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P.3.9

MEMO TO: G. F. Kemper
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Chairman, Research Committee

SUBJECT: Research Report No. 481; "Kentucky's Pavement Management System," prepared for Workshop on Pavement Management Systems; Department of Transportation; Olympia, Washington; November 8, 9, and 10, 1977

The report forwarded herewith issued from a request to participate in a workshop on pavement management systems which is being sponsored by FHWA through the Department of Transportation in the State of Washington, November 8-10, 1977. Presumably, the request stemmed from our recent report to the Fourth International Conference on the Structural Design of Asphalt Pavements. Here, we have brought forward and combined certain items from that report and from another report (issuance pending) simplifying the systems approach employed in the R-R-R study of last December. Other information is brought into convergence in a way that, hopefully, will prove helpful and meaningful there and here.

Respectfully submitted,

Jas. H. Havens
Director of Research

Enclosures
cc's: Research Committee

Research Report
481

KENTUCKY'S PAVEMENT MANAGEMENT SYSTEM

by

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and

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DEPARTMENT OF TRANSPORTATION
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KENTUCKY'S PAVEMENT MANAGEMENT SYSTEM

EXECUTIVE SUMMARY

Pavement management concepts are discussed, and a method is presented illustrating the required data and its use to accumulate EAL's annually for comparison with the design EAL. This method can be used to determine overlay priorities, overlay design thicknesses, and financing schedules. A discussion of automatic feedback of field data is presented. Pavement condition reports should be analyzed separately to prevent improper adjustments in the design system due to causes of failure other than normal fatigue.

An overlay, whether for extending service life or improving skid resistance, provides an additional structural thickness and will modify the design life.

Preparation of the R-R-R cost estimates in the fall of 1976 offered the opportunity, and necessity, to implement a pavement management scheme. The methodology used to prepare the cost estimates are discussed in relation to overlay design and scheduling of resurfacing on the Interstate System in Kentucky.

There are compounding and confounding factors which may distort pavement performance and confuse a pavement management scheme based only on the structural adequacy of the pavement. Such factors and influences as D-cracking, expansive aggregates, ditching, skid resistance, roughness, and rutting are discussed.

PAVEMENT MANAGEMENT

Pavement management *(1)* is only one portion of the management of a highway or transportation system which would include such other major categories as traffic and operations, maintenance, and finance. Subsidiary functions would include overhead categories such as planning, design, materials, construction, research, etc. Others *(2-7)* have included such items as user costs and socio-economic and environmental impact. Within the concept of this paper, the above factors definitely have an influence upon transportation management and do influence roadway design. Thus, these factors have already had their effect prior to the design and construction of a new pavement or an overlay. Pavement management is defined herein to be limited to only those factors affecting the pavement structure, not the entire roadway. Similar flow charts, such as Figure 1a and b., can be made for these other major functions.

The authors recommend that all data be entered into data files but do not recommend automatic revision of existing standards. Indeed, such action may cause unwarranted, biased, and incorrect changes.

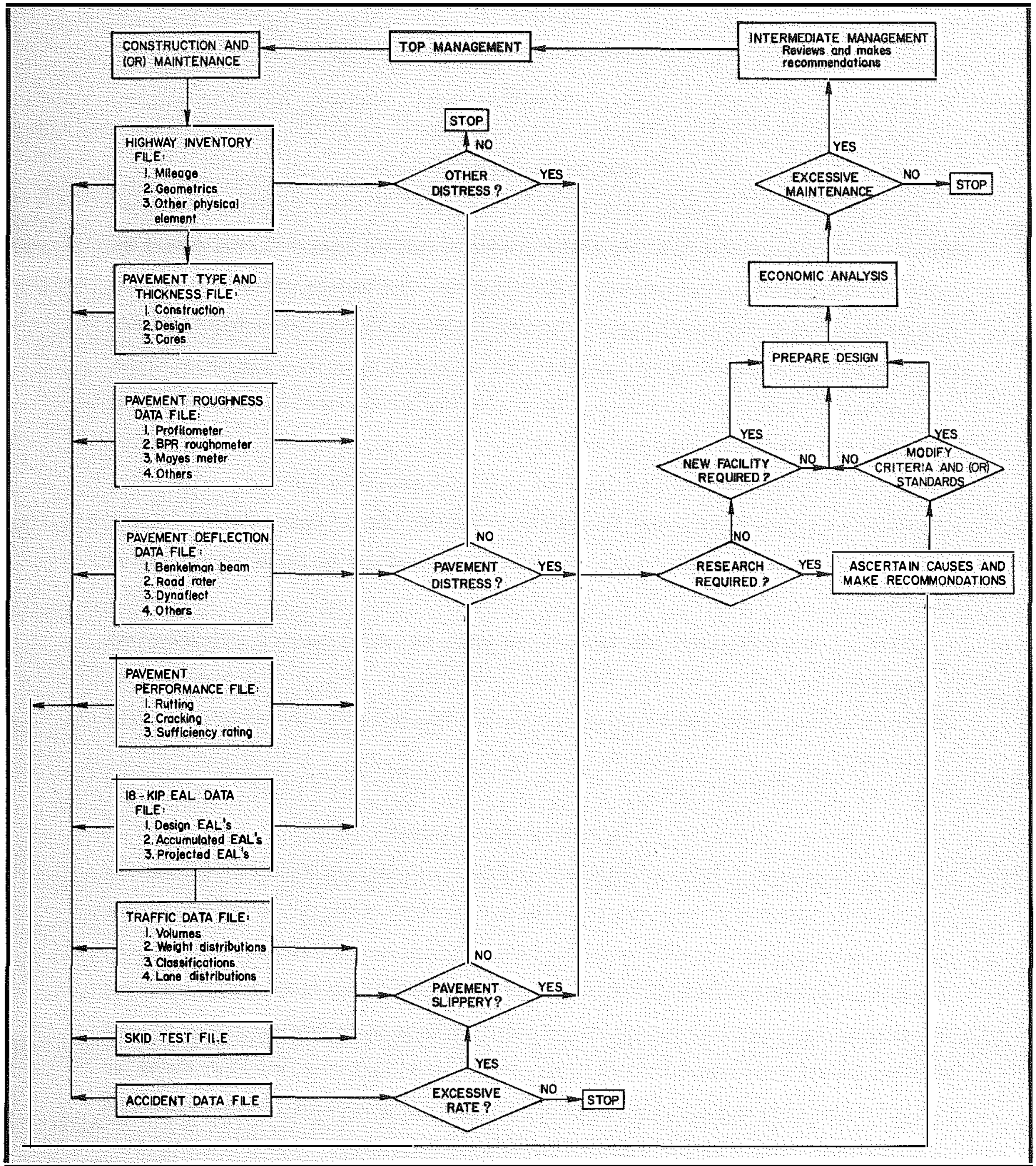


Figure 1a. Flow Diagram for Analysis of Data Files for Pavement Problems of Existing Facilities.

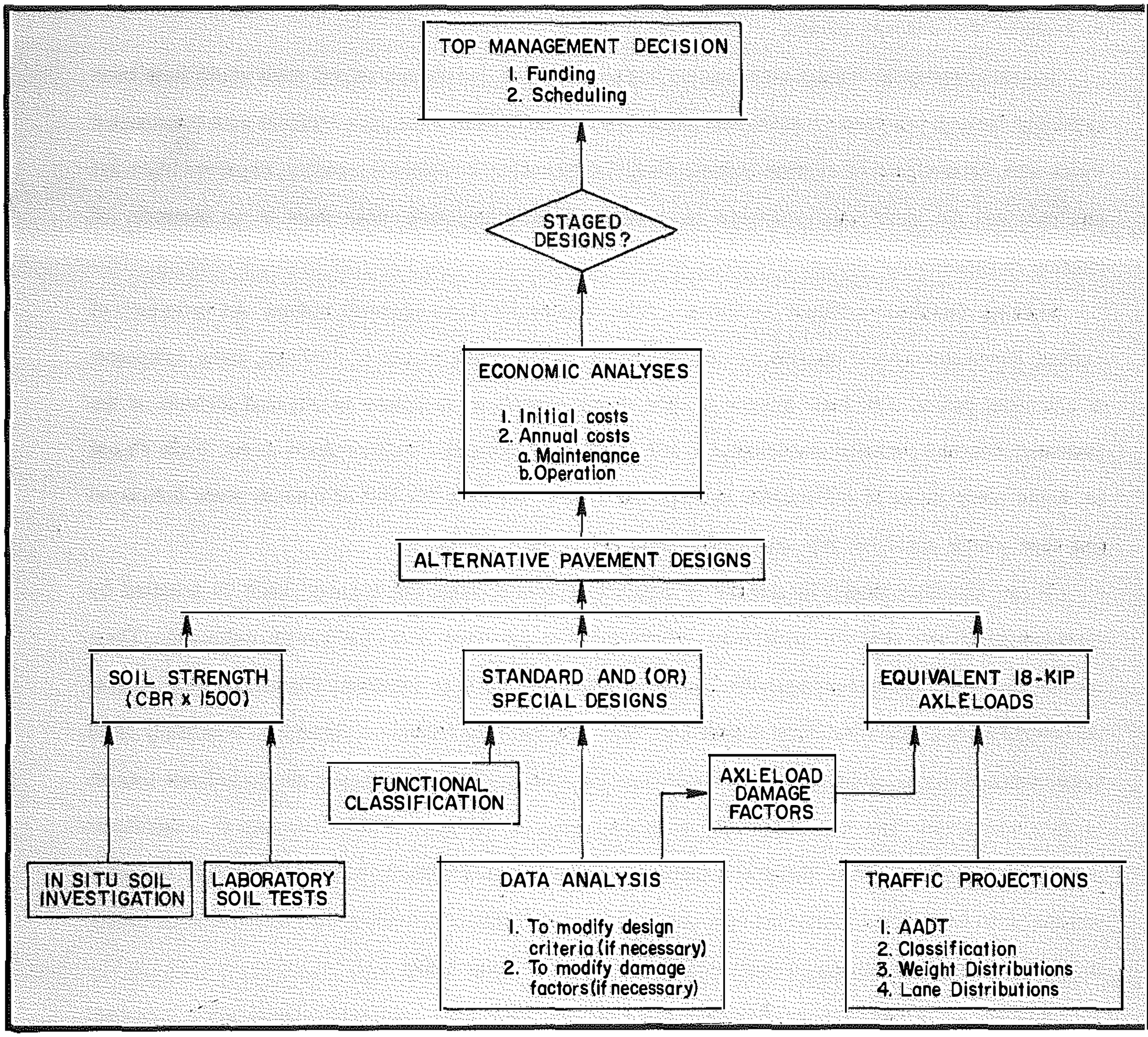


Figure 1b. Flow Diagram for Pavement Design for New Facilities.

The data should be screened to determine those which properly should become a part of the data bank to evaluate the standard criteria.

In truth, pavement management has no beginning or end. Indian and animal trails which have been expanded through the years to evolve into highways. As needs have arisen, new facilities were constructed which then became part of existing highway facilities. As society increased it mobility, improved highways emerged, and the cycle was completed once more. Thus, a beginning, now, must be an inventory of existing facilities.

Existing Facilities -- Figure 1a shows a flow diagram for a management system. An inventory of existing pavements should contain information about a predefined length of roadway, using milepoints or reference points. The basic groups of data should include an inventory of physical features of the pavement structure, surface characteristics, behavioral indicators, and traffic-related items. While Figure 1a shows many items of information and(or) test results, the data file should contain some behavioral test data which could be any one or combination of these items.

These data files should be queried at least on an annual basis, but a query could be initiated by management at any time, by inspection reports, or by a complaint from the general public, etc. These reviews can be accomplished manually but more efficiently through computer programs which analyze the computerized data files. Such a review of one data file may indicate a pavement to possibly need some remedial action. Other data files would then be reviewed for this particular site to indicate sources of data which may lead to a technically proper solution. If the same problem appears in a significant number of pavements, then there is a distinct possibility that standards may need revision. If such a revision is to be enacted, data files may need to be reanalyzed to determine the impact of such change in criteria upon the existing pavements and the resulting total needs.

Analysis of a Maintenance **Expenditures File** can have very profound effects upon pavement management and the assigning of priorities. If expenditures appear to have become excessive, two immediate possibilities exist. Either the pavement structure is in jeopardy or the existing facility is so overburdened that extensive repair is not the answer. For the latter possibility, additional lanes or a new parallel facility may be needed. Thus, the problem should be referred to planning functions for analysis. If the pavement structure is in jeopardy, then a routine overlay may not be the appropriate solution either. Figure 1a suggests an appropriate solution.

In Kentucky, research performs an important function and supplies vital input to management. Many pavement problems are routine and no longer require research. However, some repetitive problems do require research, and proposed solutions are reviewed by management. Some solutions involve answers to specific case histories. In many instances, other solutions involve proposed changes in specifications, criteria, and(or) standards. In Kentucky, management and research work together; and, in many instances, research efforts and proposals have saved unnecessary expenditures.

Assuming that standards do not need to be revised, inspection of the accumulated EAL to date, design EAL, and the current year may suggest when the design EAL could be expected to be reached. Thus, a predetermined percentage of the design EAL could be established as the indicator at which a particular pavement would be identified in a **Needs File** along with pertinent technical, economic, and calendar data, etc. for scheduling an overlay.

A pavement can become slippery for many reasons, such as abrasion under traffic, excessive loads, high asphalt contents and(or) low voids content, polishing aggregates, etc. Analyses of an **Accident File** indicate high accident-rate areas, which, when coupled with traffic volumes, may be abnormally high and may suggest the need for skid-testing. Whatever action is recommended for an existing facility is then subjected to an economic analysis which is submitted with the technical data to intermediate or top managment for consideration. This action will be discussed later.

New **Facilities** -- Figure 1b shows a flow diagram for new facilities, which are the result of two actions. The first results from an overcrowded facility which cannot possibly perform its intended function. The second comes from the initiation of a totally new system, such as the creation of parkways or the statute which authorized the interstate system. Other more subtle actions can be the result of changes in design criteria, standards, and traffic parameters such as changes in style of cargo hauler and changes in vehicle classifications of the traffic stream.

Naturally, proper design must be predicated on soil conditions over the project length and the expected traffic. Most often, traffic is the least predictable and causes the most variation between design and actual life in terms of years. The simplest process is to estimate the current, first-year AADT and to expand this value through the term of years by the compound interest equation. The compounding rate, usually between five and six percent, may be updated periodically and by class of highway. Very little information is available about the accuracy of these or other forecasts. A cursory analysis of approximately 40 projects in 1959 *(8)* indicated that 15.9 percent of the projects did or would reach the estimated, 10-year, equivalent loadings in 6.8 years. The standard deviation was 68 percent. This example does not necessarily impute the method of making forecasts but implies that if an average, compounding rate is used, the forecast will also represent an average situation. However, three standard deviations indicate a high improbability that the actual outcome of a particular project would exceed twice or be less than half the estimate.

Economic Analysis -- Figure 1c illustrates the major factors that influence an economic analysis of recommended technical action issuing from Figures 1a and b.

Management -- Figures 1d illustrates the management process which usually will include both intermediate and top levels. Intermediate management will decide which recommended actions should be considered for final action and those which should be placed in a **Schedule for Future Needs File.** The latter group would include those pavements which have reached a critical level of the designed fatigue life and should be scheduled for action during some recommended year.

Factors which bear upon decisions of top management include the submitted list of recommended actions by intermediate management, designs for new facilities, functional classification of each recommended action, available funds, and political considerations of all types. This list is then subjected to a dynamic programming analysis *(9)* which indicates those projects that will produce the highest benefit-cost ratio coupled with needs. Top management then assigns priorities, matches the funds, and the construction process begins. Those projects not funded should be immediately analyzed to determine if some possible future complication compels a rejected project to be reconsidered. Upon reconsideration, priorities are reassigned, or the project is placed in a **Needs File** for future action. Upon construction, pertinent data are added to the appropriate data files of existing facilities, thus completing the cycle.

The **Needs File** should be one of the most important files and sources of information for all levels of management. All pavements requiring overlays for any reason, the rebuilding or expansion of any existing facility, and scheduling of new constructon should be contained in this file. Since Kentucky has more than five functional classifications of highways, proposed actions should be sorted into their respective classifications. Inquiries to this file could be made for many reasons, some of which are

(1) backlog of individual needs and each respective cost estimate,

(2) backlog of cost and effort within each functional classification,

(3) information for normal budgetary efforts, and

(4) backgound for financial requests submitted to the legislature for possible changes in tax laws and funding programs.

IMPLEMENTATION OF PAVEMENT MANAGEMENT

In the fall of 1976, the Kentucky Bureau of Highways had the opportunity to implement a pavement management system, at least in part, in preparing its submittal of the R-R-R cost estimates. An objective of the 1976 Federal-Aid Highway Act *(10)* was to obtain from the states and FHWA field offices estimated costs for R-R-R (Resurfacing, Restoration, and Rehabilitation) work necessary to maintain the high level of transportation service intended to be provided by the Interstate System. The Secretary of Transportation, US Department of Transportation, was required to report to Congress by May 5, 1977.

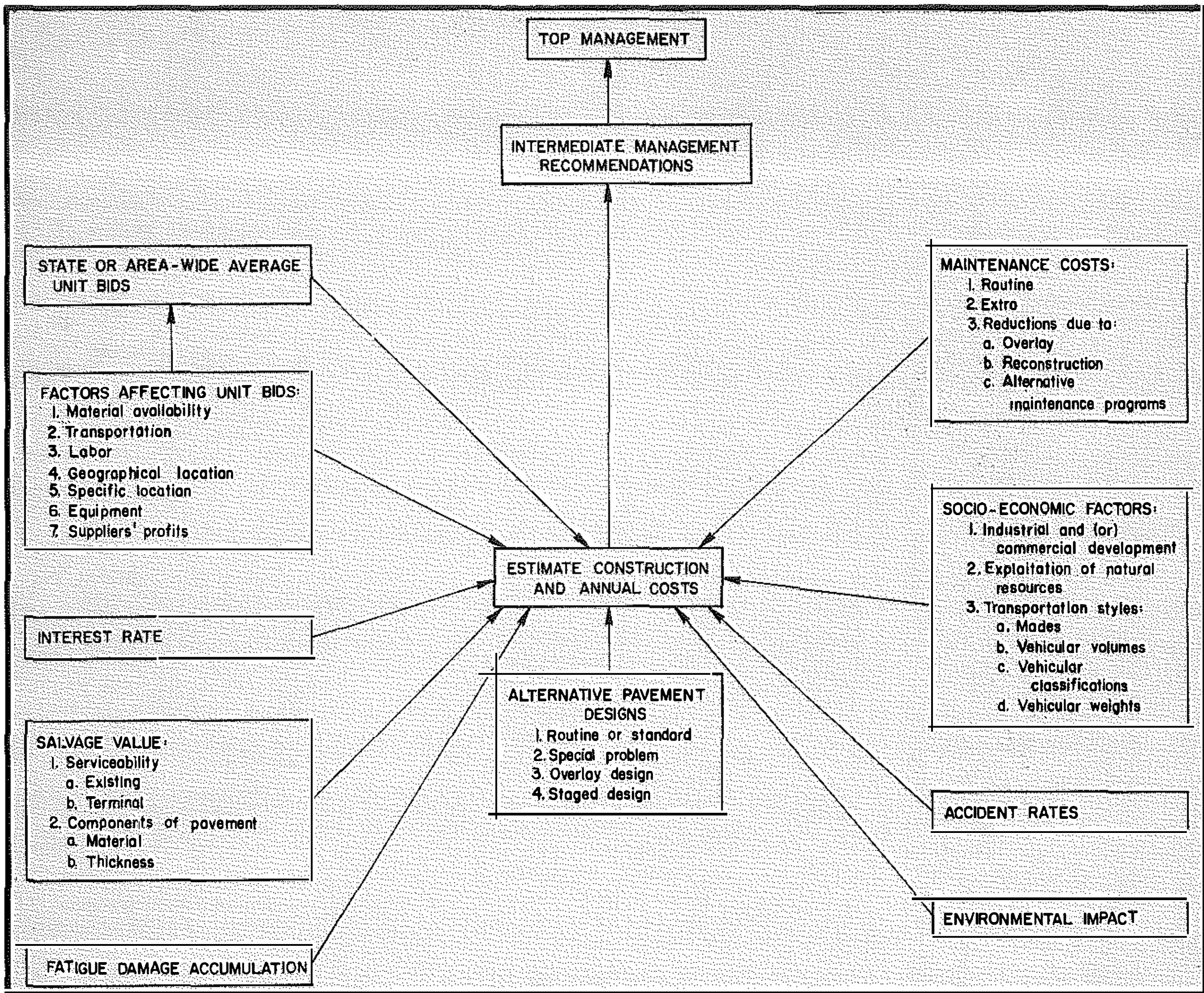


Figure 1c. **Flow Diagram for Economic Analysis of Recommended Action for Existing and New Facilities.**

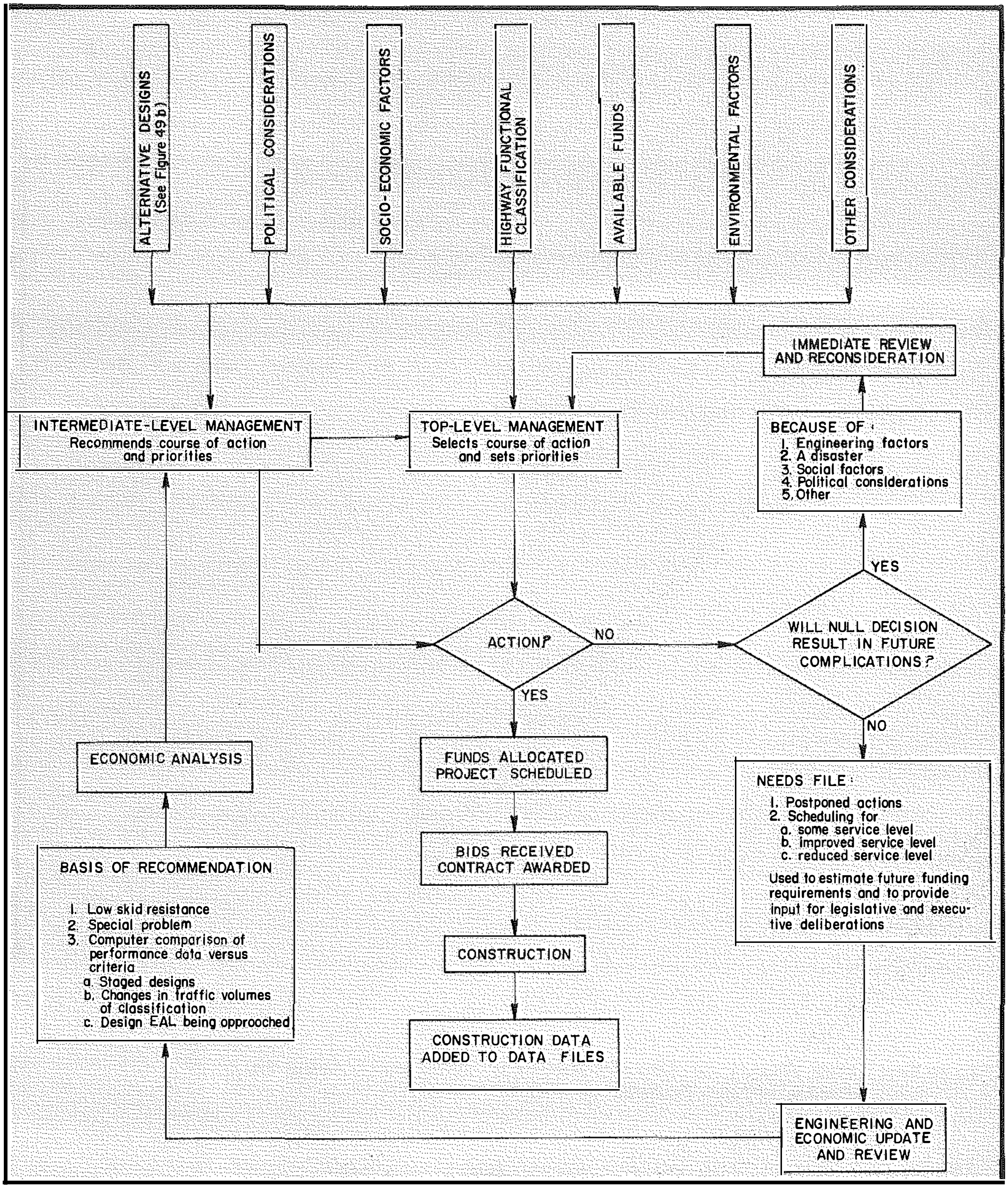


Figure 1d. **Flow Diagram for Intermediate and Top Management Decisions Related to Pavement Management Problems.**

The Present Serviceability Index (PSI), as determined from road roughness, is an index or measure of the serviceability of a pavement and was used in the study to indicate the need for resurfacing, restoration, and(or) rehabilitation. The PSI is a function of the as-constructed roughness, changes in roughness as a function of time, subgrade settlements or expansions, and effects of traffic. Thus, historical data to be analyzed included pavement roughness measurements, traffic volumes and classifications, and axleloads and distributions; those data were used for fatigue-type analyses of past and future conditions.

Pavement Roughness Data -- The Division of Research already had a data file of pavement roughness for most of the interstate system. Data were by lane and direction, date of test, and date opened to traffic. A graph (see Figure 2 for an example) was made for each contract section, and an equation was fitted to the data by linear regression. Equations used were of the form

$$y = m x + b \qquad 1$$

or

$$y = m \log x + b, \qquad 2$$

where y = roughness test value,

x = months opened to traffic, and

m and b = constants.

A slope of zero and an intercept of 100 was assigned to contract lengths for which roughness tests had not been made.

Traffic Volume Data – AADT's from traffic volume maps were converted to one-directional AADT by dividing by 2. The reduced values were plotted versus year (see Figure 3 for an example). For most contract sections, equations fitted to the data by linear regression were of the form

$$\log AADT = m \log (year - 1960) + b \qquad 3$$

where AADT = one-way volume and m and b = constants.

Near urban areas, the AADT could be relatively large. Visual inspection of AADT plots sometimes showed two distinct patterns of increasing AADT's. As traffic volumes approached the capacity of the section of highway under consideration, the rate of growth of AADT appeared to approach zero. Thus, Equation

Figure 2. Roughness versus Time (Months).

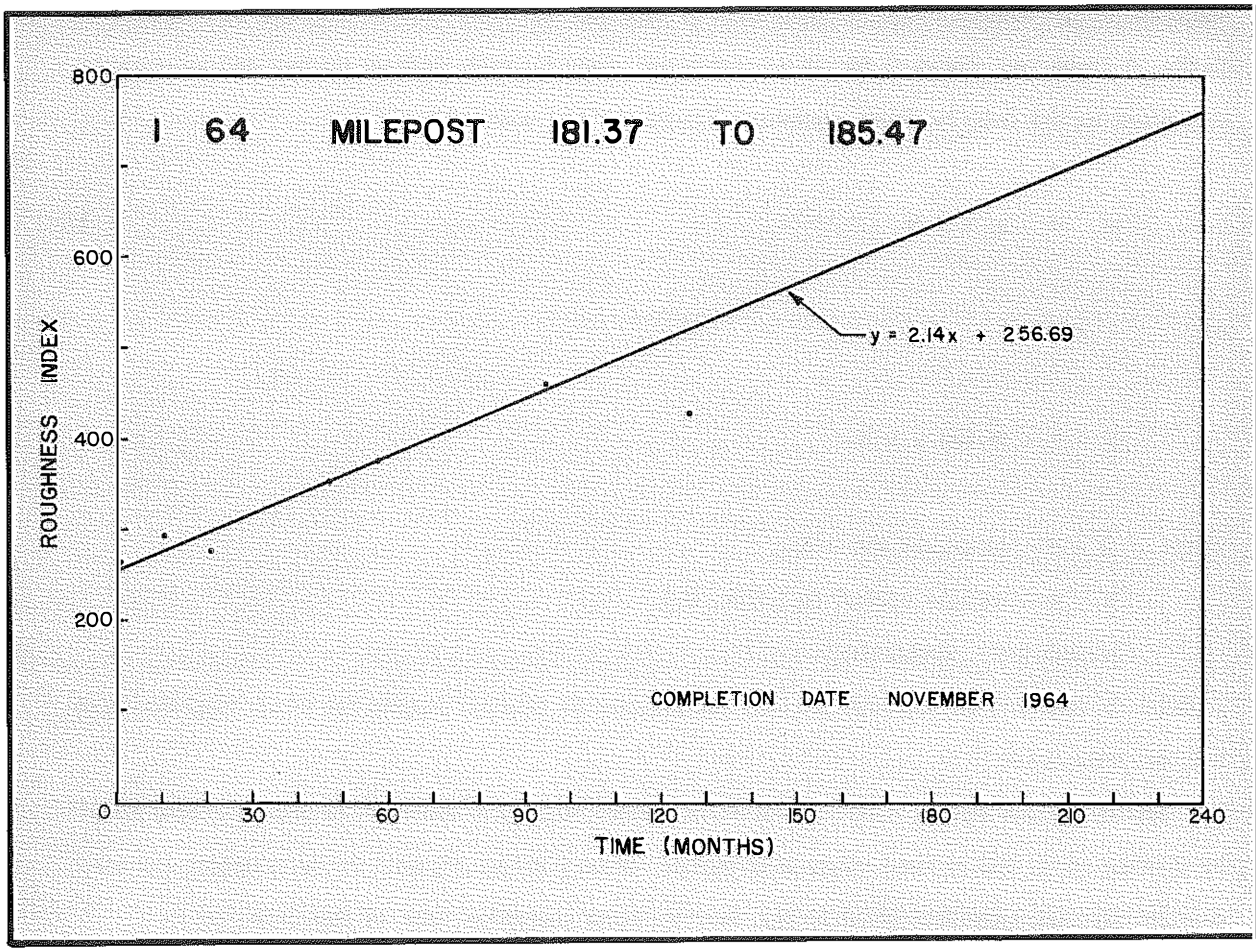
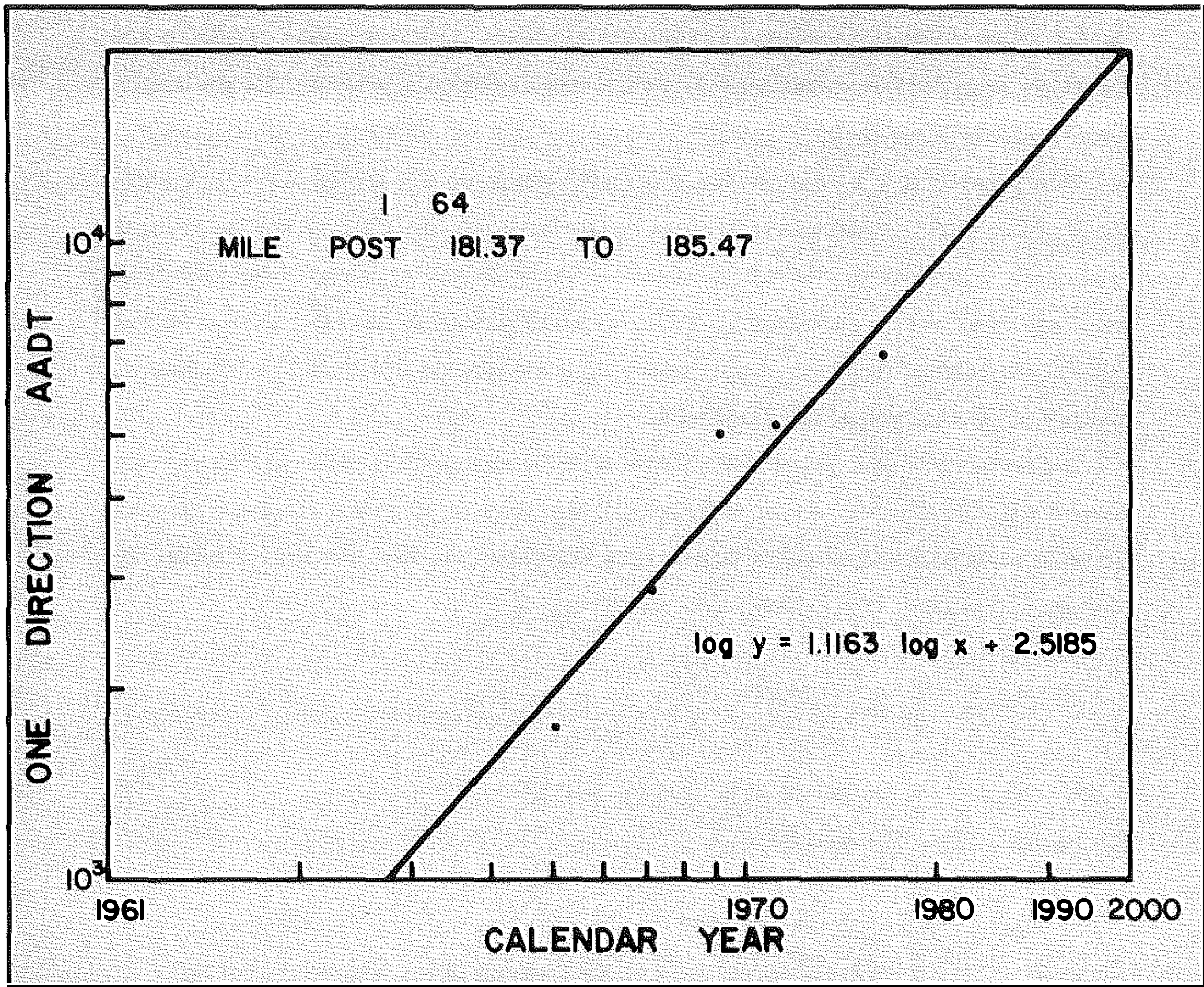


Figure 3. AADT versus Calendar Year.



3 had to be applied in two stages where the AADT had already approached the capacity and could not sustain the same growth rate through 1995. This procedure was applied to each interchange-to-interchange length.

Vehicle Classifications -- The types and numbers of vehicles in the traffic stream have significant influence upon the fatigue life of pavements. The Division of Research had developed a method to estimate the percentages of vehicle classifications as a function of one-way AADT and number of lanes. Figure 4 combines results from several reports *(1, 11, 12, 13)* and incorporates subsequent traffic classification volume counts into one graph.

Lane Distributions -- A study *(13)* had shown that approximately 90 percent of the truck traffic and 73 percent of the automobiles and pickup trucks used the shoulder lane of a rural, four-lane interstate highway. For a new six-lane interstate, 48 percent of the trucks and 41 percent of the automobiles and pickup trucks used the center lane. For I 75 in northern Kentucky, the shoulder lanes of the six-lane sections were analyzed for fatigue because these sections orginally had four lanes; thus the historical, maximum fatigue occurred in the shoulder lane.

Damage Factors -- Truck weight studies *(12)* provided one source of data. An average damage factor for each vehicle classification, as given in Table 1, had been developed for flexible pavements. The 1971 Kentucky modifications *(1)* to the AASHTO equivalencies *(14)* were used. For single axleloads, the equation for equivalent 18-kip (80-kN) axleloads (EAL_{18}) is

$$EAL_{18} = N_{sa}(1.2504)^{(P - 18)} \qquad 4$$

where N_{sa} = number of single axleloads of P kips.

For tandem axleloads, the equation is

$$EAL_{18} = N_{ta}(1.1254)^{(P - 34)} \qquad 5$$

where N_{ta} = number of tandem axleloads of P kips.

For PCC pavements, the Kentucky modified AASHTO equation for single axleloads is

$$EAL_{18} = N_{sa}\ (1.2875)^{(P - 18)} \qquad 6$$

where N_{sa} = number of single axleloads of P kips.

Figure 4. AADT versus Vehicle Classification Percentages.

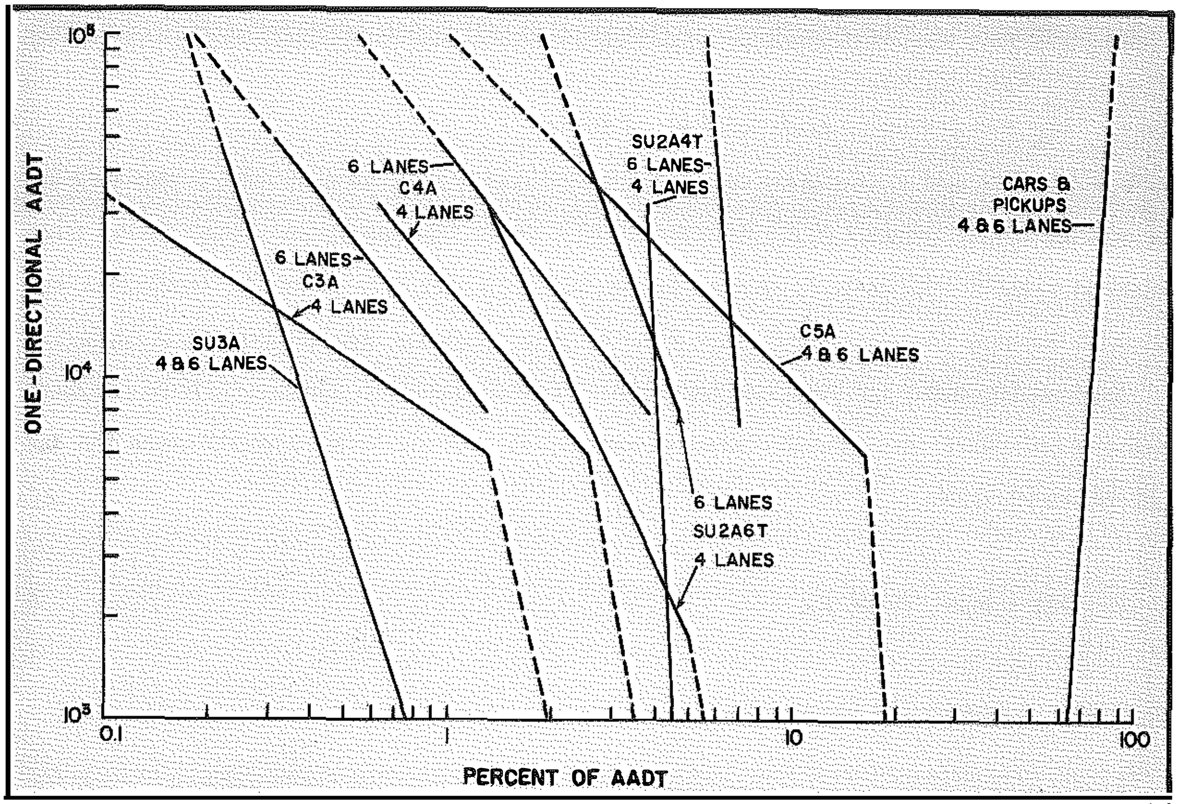


TABLE 1. DAMAGE FACTORS BY VEHICLE CLASSIFICATION

VEHICLE TYPE	NUMBER OF WEIGHED VEHICLES	TOTAL EQUIVALENT 18-KIP (50-kN) LOADS	FLEXIBLE PAVEMENTS AVERAGE 18-KIP (80-kN) LOADS PER VEHICLE	RIGID PAVEMENTS DAMAGE FACTOR* BY YEAR m	b
SU2A4T: Single-Unit, 2 Axles, 4 Tires	8,564	518.2	0.0605	0.008163	-3.33130
SU2A6T: Single-Unit, 2 Axles, 6 Tires	19,058	5,627.6	0.2953	0.001291	-1.31031
SU3A: Single-Unit, 3 Axles	2,848	1,818.7	0.6386	0.021921	-2.39605
C3A: Combination, 3 Axles	4,701	2,986.7	0.6353	0.009690	-1.10220
C4A: Combination, 4 Axles	15,217	11,434.7	0.7514	-0.021267	1.07675
C5A: Combination, 5 Axles	21,673	13,583.1	0.6267	0.018606	-1.78483
Automobiles and Pickups			0.0501	0.00	-2.99373

NOTE: Data from Kentucky W-4 tables for years 1959-1973, except for Automobiles and Pickups. Average EAL per Automobile and Pickup obtained from Division of Research data.

*Damage Factor (Year) = m x (Year - 1959) + b

For tandem axleloads, the equation is

$$EAL_{18} = N_{ta}(1.1500)^{(P - 29)} \qquad 7$$

where N_{sa} = number of single axleloads of P kips.

Truck weight data used in developing damage factors for each type of truck in Table 1 were used for the portland cement concrete (PCC) pavement analysis except that appropriate damage factors from Equations 6 and 7 were used. An average damage factor for each type of vehicle classification was calculated for each year. Trend lines were fitted by linear regression and are included in Table 1.

Serviceability Index and Pavement Roughness -- Previous research in Kentucky *(15)* on pavement roughness had shown the correlation between roughness, RI, and present serviceability index, PSI, for rigid pavements to be

$$PSI = 6.01 - 0.006\ RI; \qquad 8$$

for flexible pavements, the relationship is

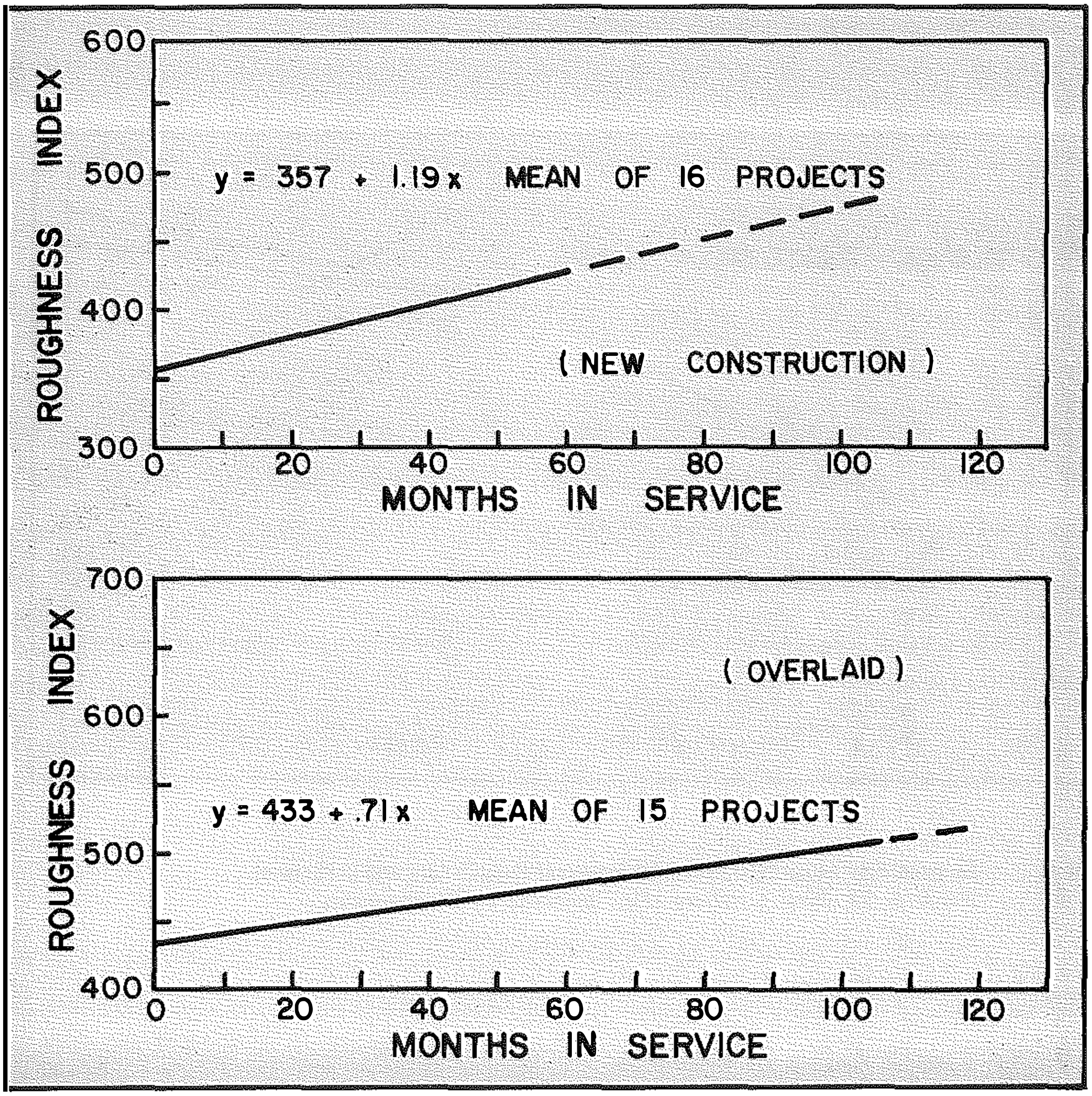
$$PSI = 4.65 - 0.003\ RI. \qquad 9$$

For the R-R-R cost estimate, it was assumed that a pavement would begin to show signs of distress, faulting, rutting, cracking, etc. at a time when the process of scheduling resurfacing should begin. It was assumed that actual paving operations would begin by the time the PSI reached 3.25, which correlated to a RI of 460 for both PCC and asphaltic concrete (AC) pavements.

Rizenbergs, et al. *(15)*, correlated months of service with RI for newly constructed and overlaid pavements. It was noted that the slope of the correlation for overlaid pavements was approximately 0.6 that of newly constructed pavements (Figure 5).

Serviceability Index, Designed Fatigue Life, and Existing Structural Worth -- Experience with Kentucky pavements has shown that interstate routes have sufficient pavement thicknesses to minimize rutting and fatigue cracking. Thus, the designed fatigue life is "consumed" even when the pavement structure still has a relatively high serviceability index -- especially when compared to other highway systems, such as rural secondary routes. Figure 6a illustrates the concept of newly constructed PCC

Figure 5. Roughness Index versus Time; Newly Constructed and Overlaid Pavements.



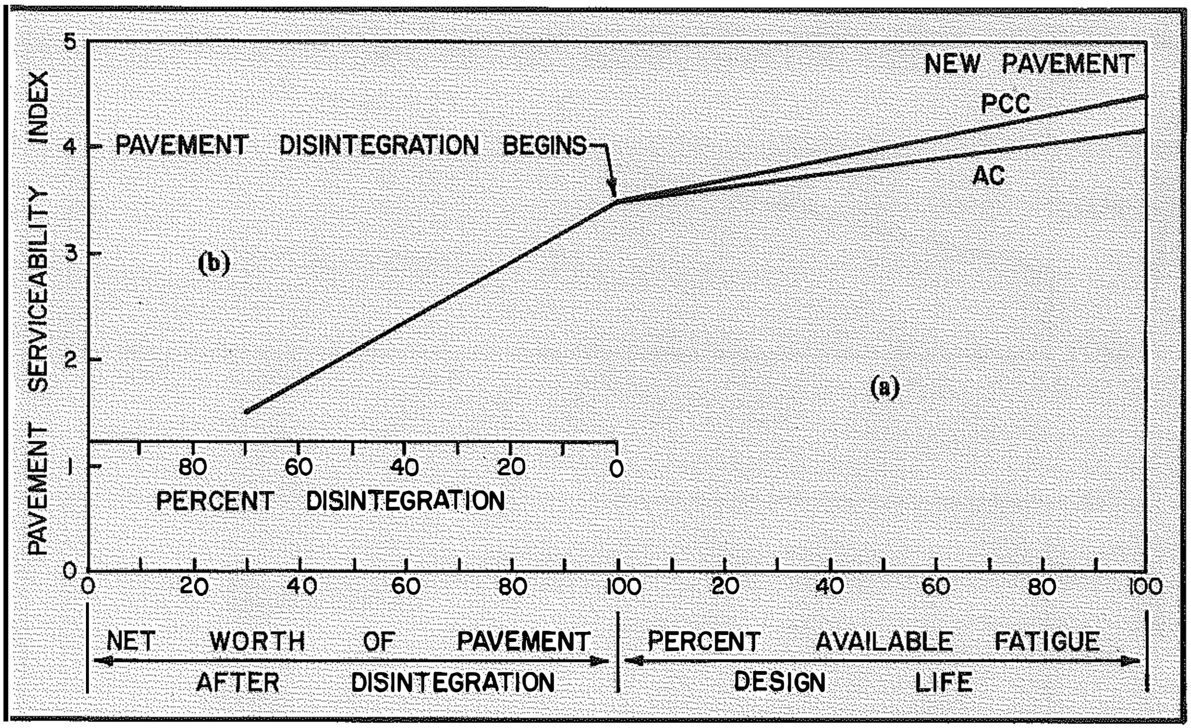


Figure 6. Serviceability Index Related to (a) Designed Fatigue Life and (b) Present Worth of Pavement Structure after Beginning of Disintegration.

and AC pavements having serviceability indices of 4.5 and 4.2, respectively. When the designed fatigue life has been consumed, disintegration of the pavement structure has been observed to begin, and the pavement structure at this point in time has been assumed to have a PSI of 3.5.

Figure 6b illustrates the concept of the percent of present worth of the pavement structure after disintegration begins. Disintegration begins at the end of the designed fatigue life. The pavement structure is assumed to be worth only 30 percent when the PSI reaches 1.5.

Estimation of Historical and Future 18-kip (80-kN) EAL's -- A computer program was written to utilize interchange-to-interchange lengths, equations fitted to roughness measurements, equations fitted to trends of AADT versus year, vehicle classifications as a function of AADT and number of lanes, lane distribution factors, and axleload damage factors from Table 1. The output listed the route, beginning and ending mileposts, month and year opened to traffic, calendar year, estimated 18-kip (80-kN) EAL's, and estimated roughness.

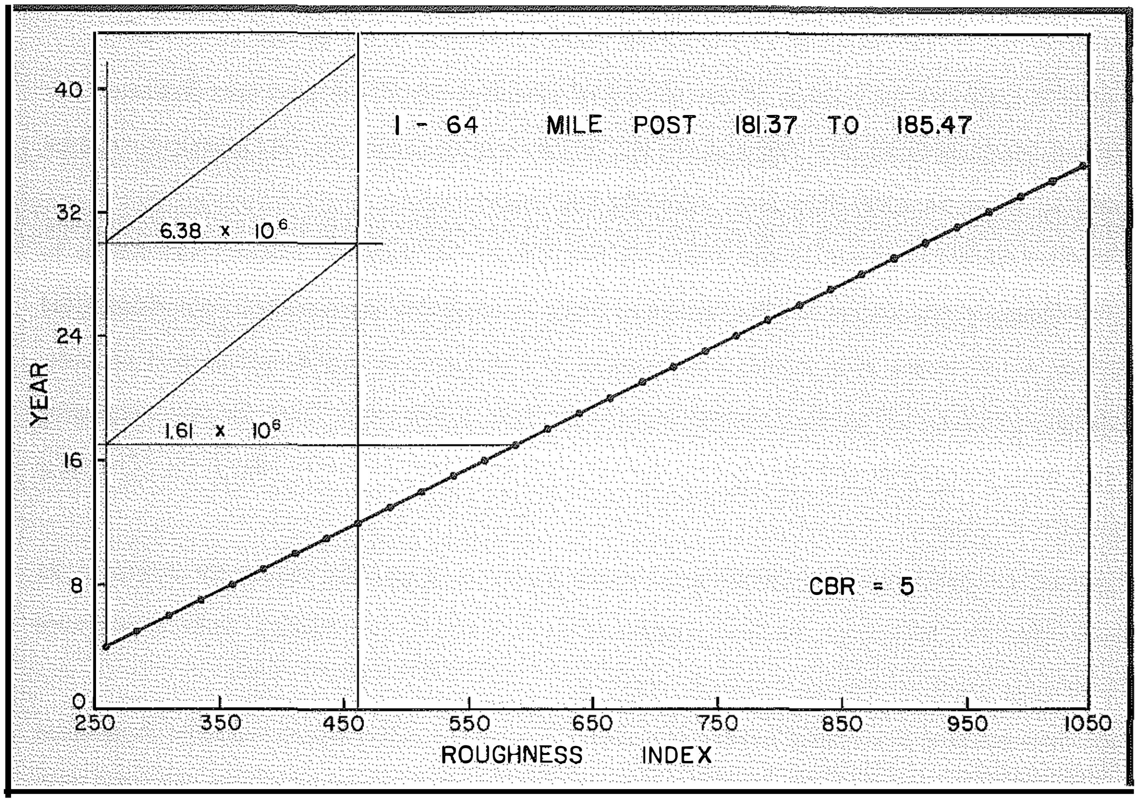
A second computer program utilized the Calcomp plotter to graphically display the estimated RI and 18-kip (80-kN) EAL's from the first program (Figures 7 and 8, respectively). These graphs were used to estimate when an overlay would be required.

Each interchange-to-interchange length was analyzed. The year an initial overlay was to be constructed was determined to be the year the RI value reached 460, or 1977 if the RI already exceeded 460. It was assumed the new overlay would have a RI no less than the RI of the original construction. The number of years from the date of opening to the year the RI reached 460 was divided by 0.6 to determine the number of years when the RI again would reach 460 and the second overlay would be required. The same interval of time between the first and second overlays was used to determine when the third overlay would be required.

DESIGNS OF OVERLAYS

Overlays for Rigid Pavements -- Westergaard's equation for corner loading was used to obtain an allowable working stress of 210 psi (1.4 MPa). The Portland Cement Association's single-axle design *(16)*, using 210 psi (1.4 MPa) as the allowable working stress, a subgrade reaction modulus "k" of 150 pci (4.2 Mg/m^3), and an 18-kip (80-KN) axleload, yielded an 8-inch (200-mm) thickness. A load-repetitions relationship was developed for a terminal serviceability of 2.5 using the AASHTO equation *(14)*. Stresses and strains computed from the Chevron N-layered program were used to develop a relationship between stress ratio and repetitions to failure (Figure 9). At a 210-psi (4.8-MPa) modulus of rupture for concrete, the stress ratio was computed as 210/700 = 0.30. At this stress ratio, Figure 9 yields 11 x 10^6 repetitions of an 18-kip (80-kN), single axleload.

Figure 7. Calendar Year versus Roughness Index.



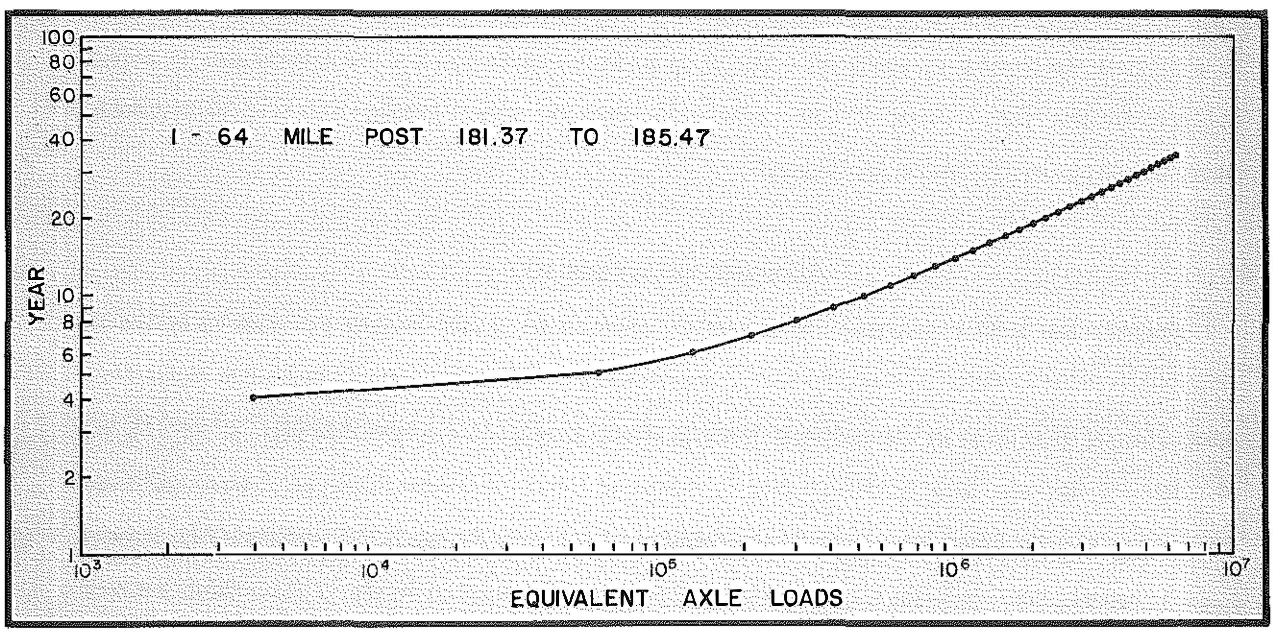
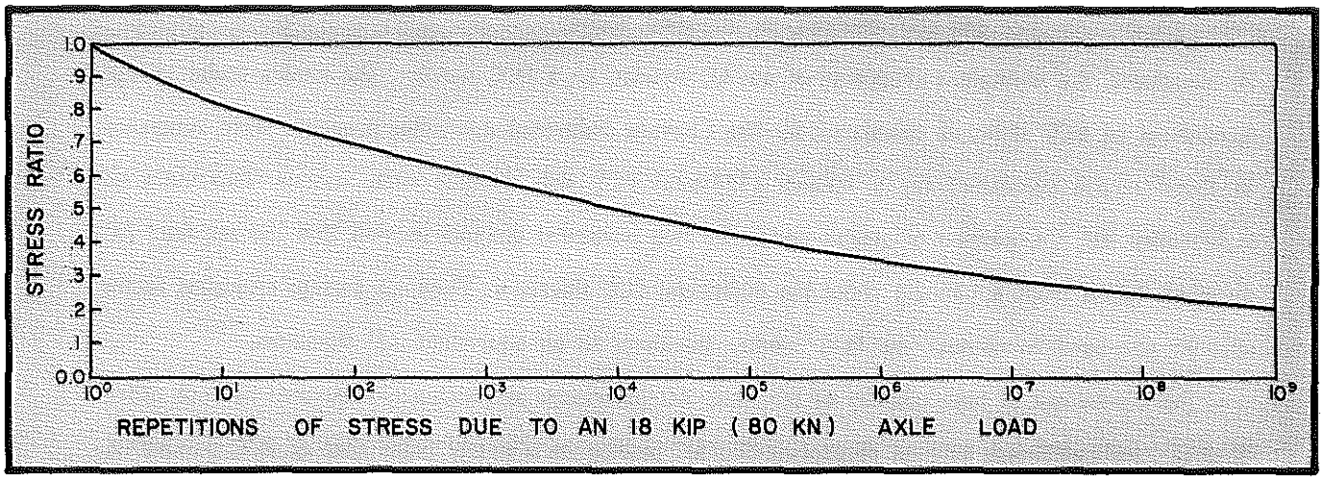


Figure 8. Calendar Year versus Accumulated 18-kip (80-kN) EAL's.

Figure 9. Stress Ratio versus Repetitions of Load; Corner Loading.



The PCC overlay thickness was determined according to the estimated 18-kip EAL's for the calendar years 1977, 1995, and those years dictated from roughness analyses by

$$x = 8\,(N_{year} - N_{1977})/11 \times 10^6 \qquad 10$$

where x = thickness of PCC overlay required (inches),

N_{year} = estimated, accumulated number of 18-kip (80-kN) EAL's through the year in question,

N_{1977} = estimated, accumulated number of 18-kip (80-kN) EAL's through 1977, and the constant 8 is the slab thickness associated with 11 x 10^6 repetitions of an 18-kip (80-kN) axleload, as discussed above.

To determine the equivalent thickness of an AC overlay to a PCC overlay, the strains under several thicknesses of PCC slabs, as calculated by the Chevron N-layered computer program, were compared to those strains calculated for a 10-inch (154-mm) PCC slab overlaid with various thicknesses of high-quality AC. Comparison of strains under the two pavement systems showed that the required AC thickness varied from 1.8 to 2.1 times the additional PCC thickness. Thus, an average empirical factor of 1.95 was used to convert the PCC overlay thickness to an equivalent AC thickness.

The coefficient of heat transfer of PCC and AC pavements are within two percent of each other. However, the coefficient of heat absorption for AC pavements is nearly twice that of PCC pavements. Temperature distribution analyses in AC pavements *(17)* indicated that at least 6 inches (150 mm) of AC is required in Kentucky to reduce the temperature of the AC at the AC-PCC interface to a value no greater than the original PCC surface temperuature without an AC overlay. Any overlay thickness less than 6 inches (150 mm) increases the probability of raising the temperature in the PCC slab, causing a greater compressive force in the restrained pavement slab and increasing the probability of blow-ups. Another 2 inches (50 mm) of AC was added to minimize the possibilities of reflection cracking and to lower the PCC temperatures to reduce the probability of increased compressive stresses. Thus, the minimum AC overlay was set at 8 inches (200 mm).

Overlays for Flexible Pavements -- To determine the effective structural worth of an existing pavement, an analysis was made of dynamic deflection test data obtained with the Road Rater on I 64 in Boyd County on March 15, 1976. The original pavement consisted of 7.5 inches (190 mm) of AC on 15 inches (380 mm) of dense-graded aggregate (DGA) base. The surface temperature was measured during testing, and the temperature distribution in the pavement for that time period was determined;

an appropriate distribution of dynamic moduli was determined from temperature-moduli relationships (Figure 10).

Sensor 1 on the Road Rater is located directly under the center of the oscillating mass. Sensors 2 and 3 are located 12 and 24 inches (300 and 600 mm), respectively, from Sensor 1. Deflections at Sensors 2 and 3 can be used to interpret the pavement behavior by using the equation

$$\text{Projected Deflection} = 10^{[2 \log \text{No. } 2 - \log \text{No. } 3]} \qquad 11$$

where No. 2 = deflection, Sensor 2, and

No. 3 = deflection, Sensor 3.

Projected deflections for I 64, Boyd County, were plotted versus readings from Sensor 1, as shown in Figure 11. The majority of the data were contained within a relatively narrow band. If the projected deflection is larger than the expected value for Sensor 1, Sensors 2 and 3 readings will be higher than normal and indicate a weak subgrade or foundation. If the Sensor 1 value equals or exceeds the projected deflection, the bound portion of the pavement may be cracked and relatively weak.

As a measure of the ability of a pavement structure to "spread" the applied load to the subgrade, the deflection readings of the first three sensors of Kentucky's Road Rater have been found to be the most meaningful. The sensor readings were used to enter the appropriate portion of Figure 12, and the subgrade moduli were obtained for the 7.5 inches (190 mm) of AC, the original thickness. The three subgrade moduli obtained were summed and averaged. Figure 13 illustrates the average of deflections versus the average subgrade modulus. The range of subgrade moduli corresponds to a range of Kentucky CBR values form 5 to 15. The mean CBR was approximately 8.

Roughness measurements indicated a relatively low serviceability index (Figure 14) and corresponded to different levels of deterioration. Weaker areas were analyzed using Road Rater readings as entrance values to appropriate portions of Figure 12 and the equivalent structure was determined to be approximately 5 inches (125 mm) of AC on 14 inches (355 mm) of DGA. Such analyses permitted the development of Figure 15.

The year in which an overlay would be required was determined from the estimated roughness index, as discussed earlier. The estimated, accumulated 18-kip (80-kN) EAL's for the same year and the length of the project were obtained from the computer output. The design CBR was used whenever it was known; otherwise, the CBR was assumed to be 5. Design charts *(1)* were entered with the design CBR and the required number of 18-kip (80-kN) EAL's for that year. The total thicknesses were determined and labeled as Curve A in Figure 16.

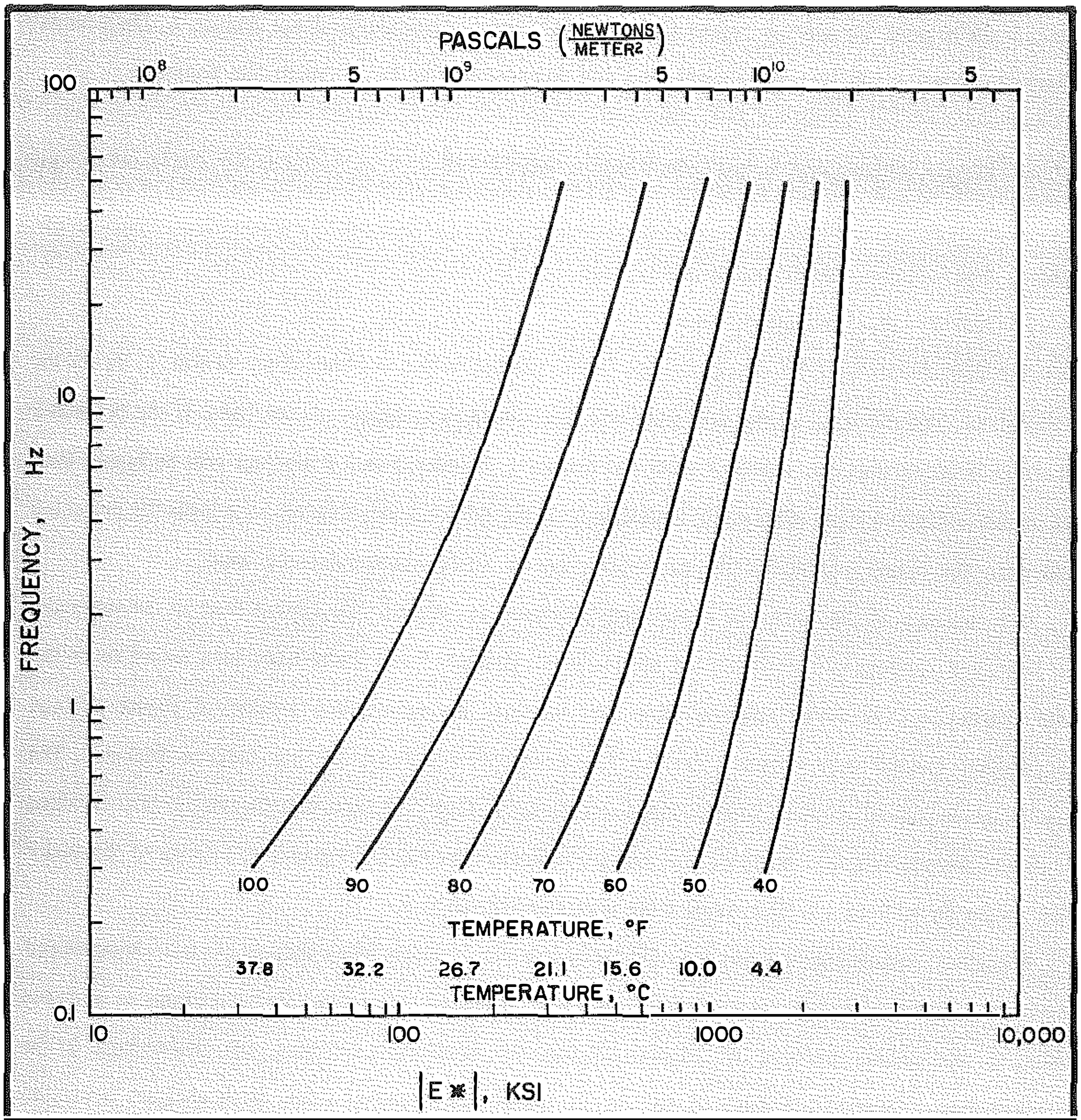


Figure 10. Influence of Temperature and Frequency upon Dynamic Modulus of Elasticity of Asphaltic Concrete.

Figure 11. Road Rater Data; I 64, Boyd County.

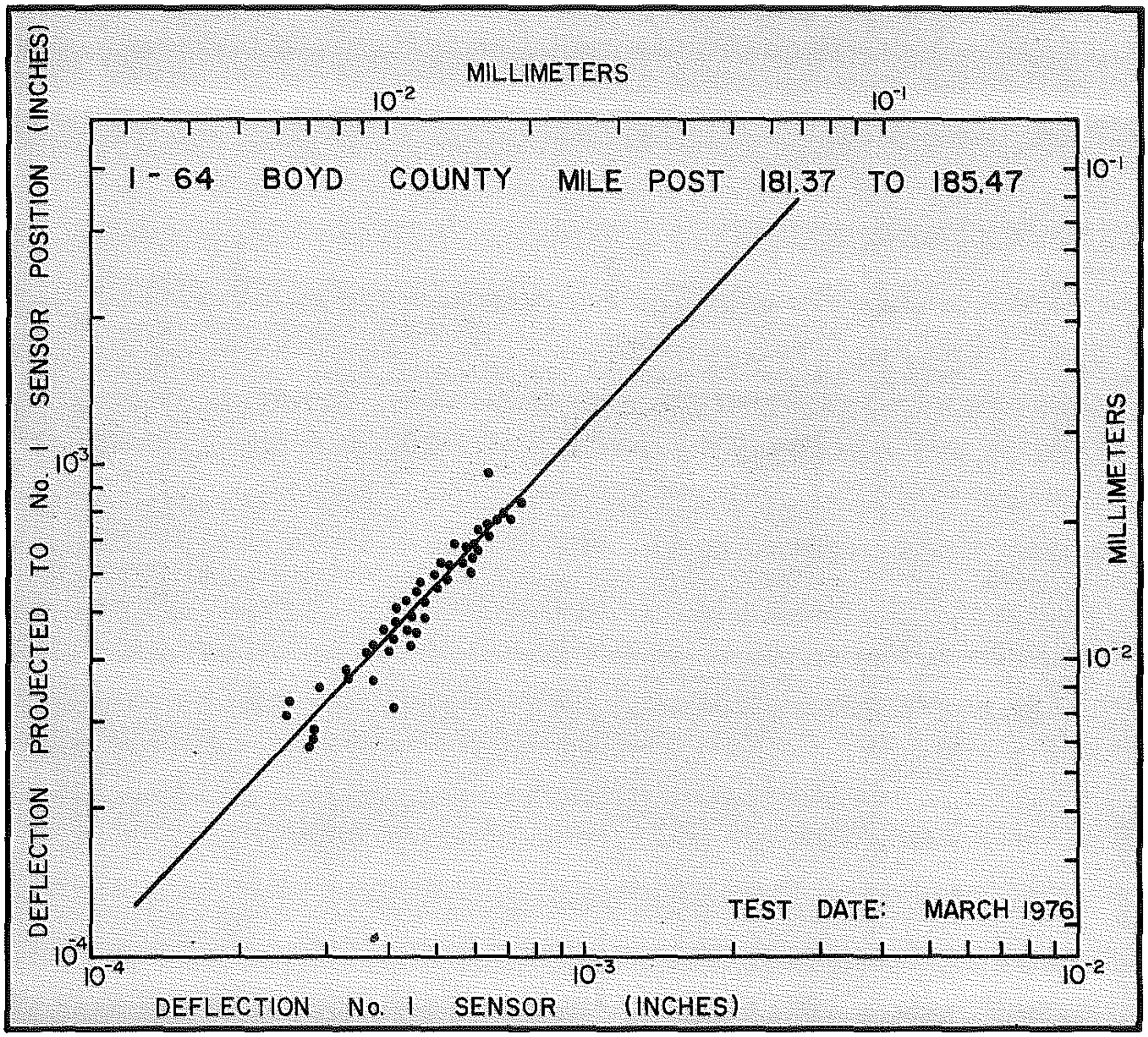
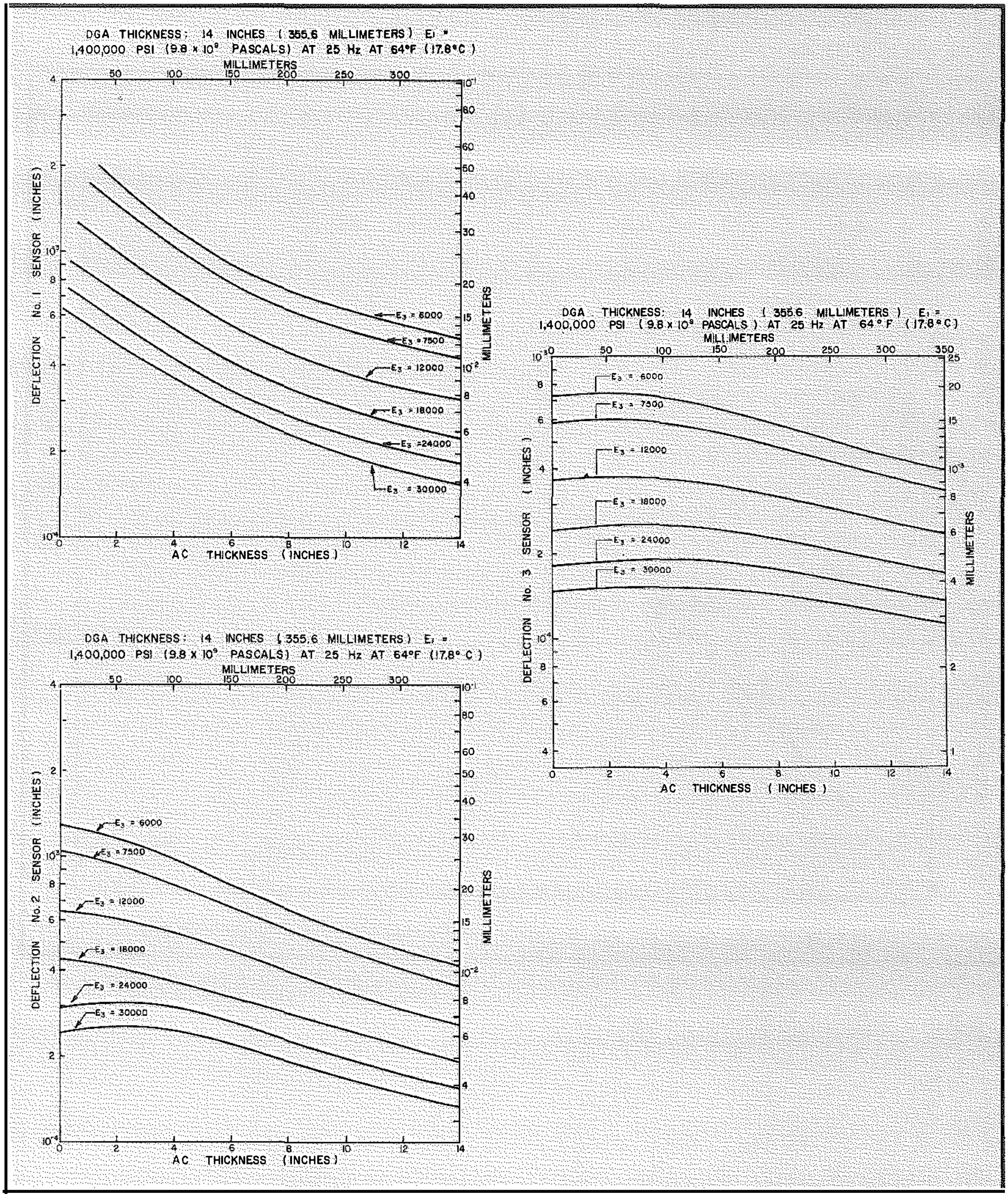


Figure 12. **Road Rater Deflection versus Thickness of Asphaltic Concrete; 14 Inches (355 mm) of DGA, for Asphaltic Concrete Modulus of 1,400,000 psi, (9.7 GPa) and Various Subgrade Moduli.**



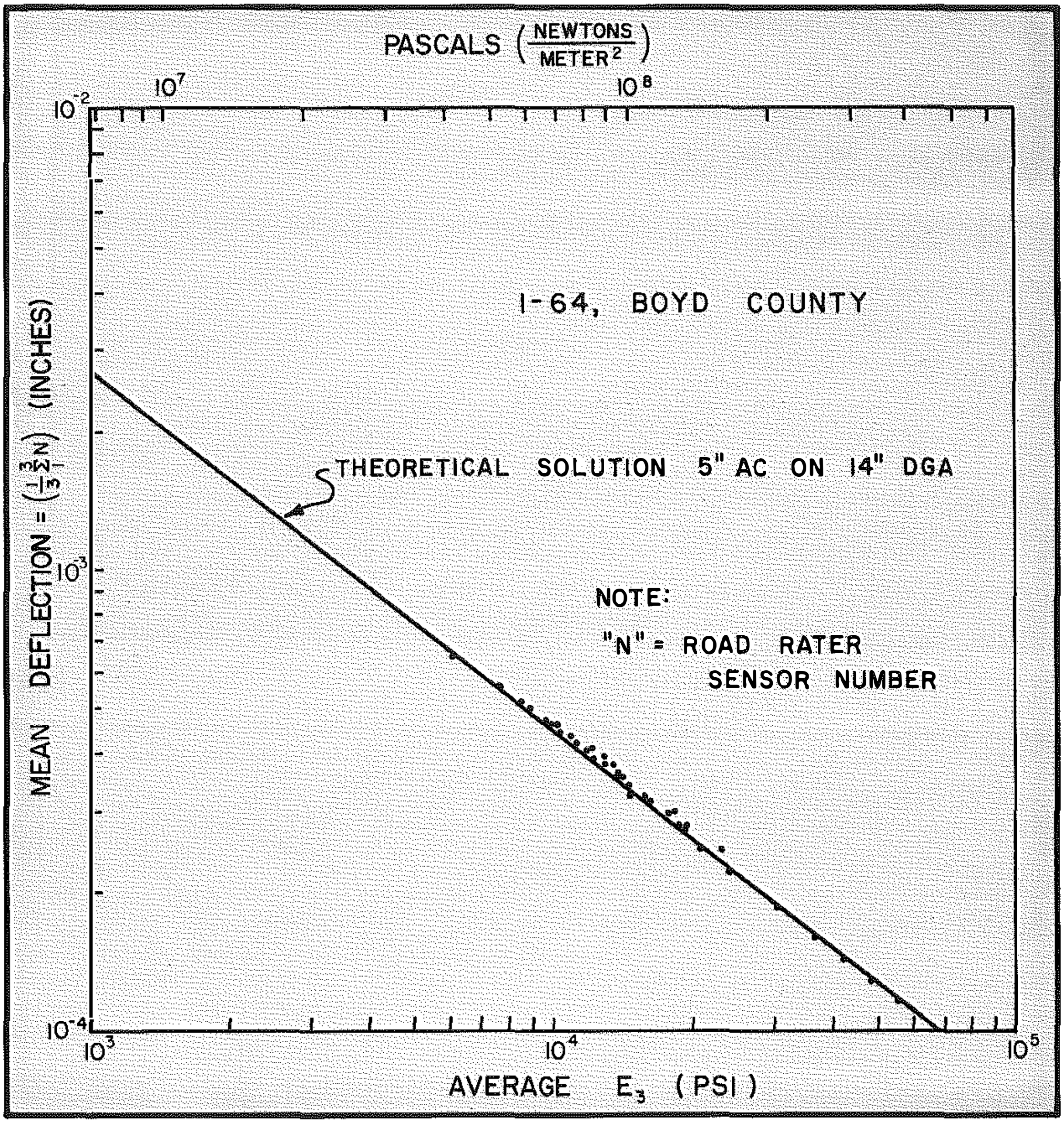
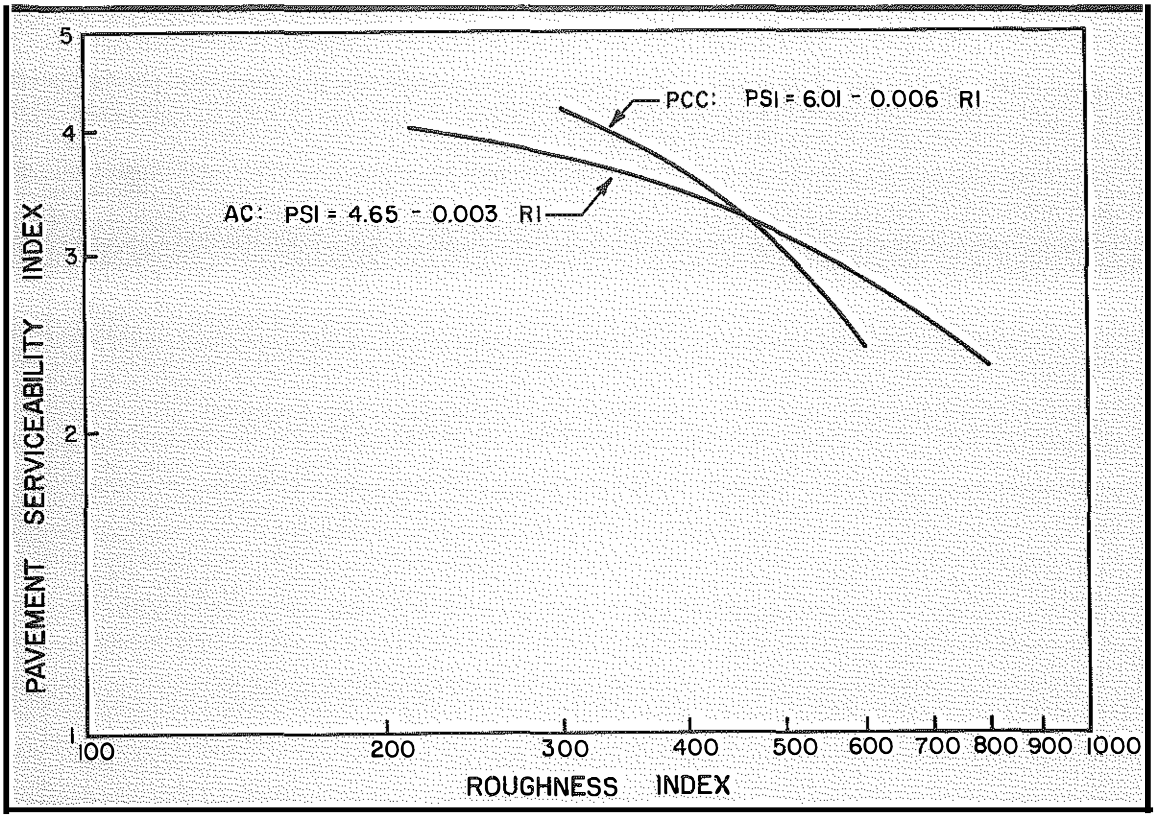


Figure 13. Average Dynamic Deflections and Averaged Subgrade Moduli; I 64, Boyd County.

Figure 14. Serviceability Index versus Roughness Index.



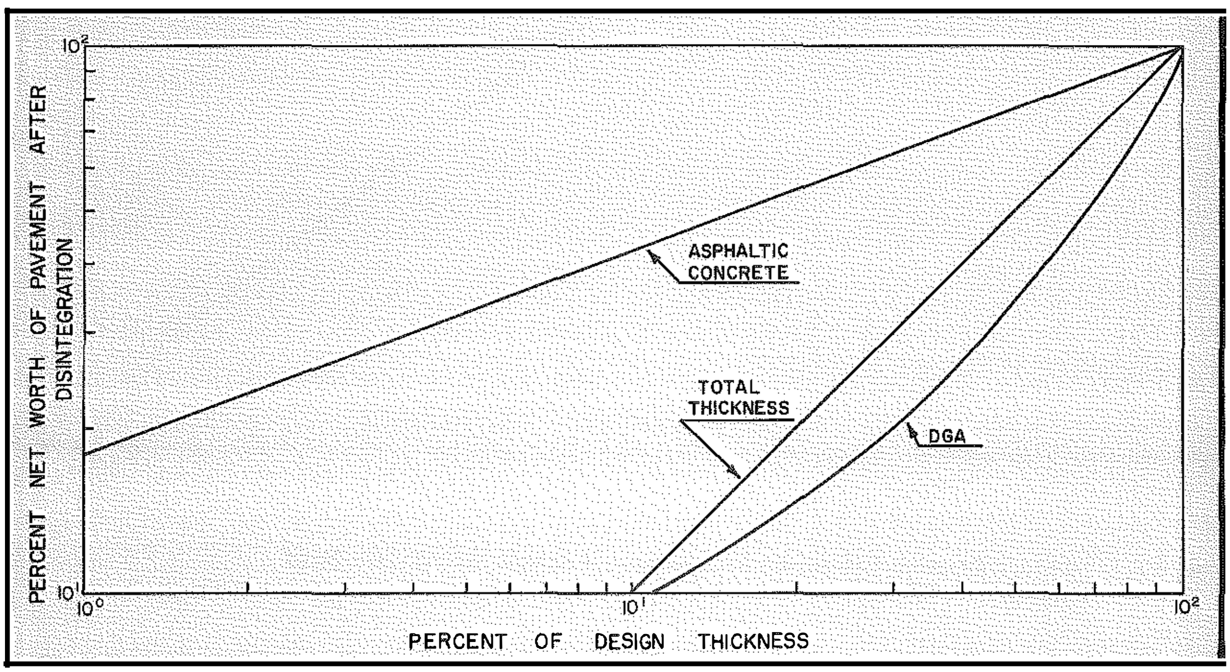
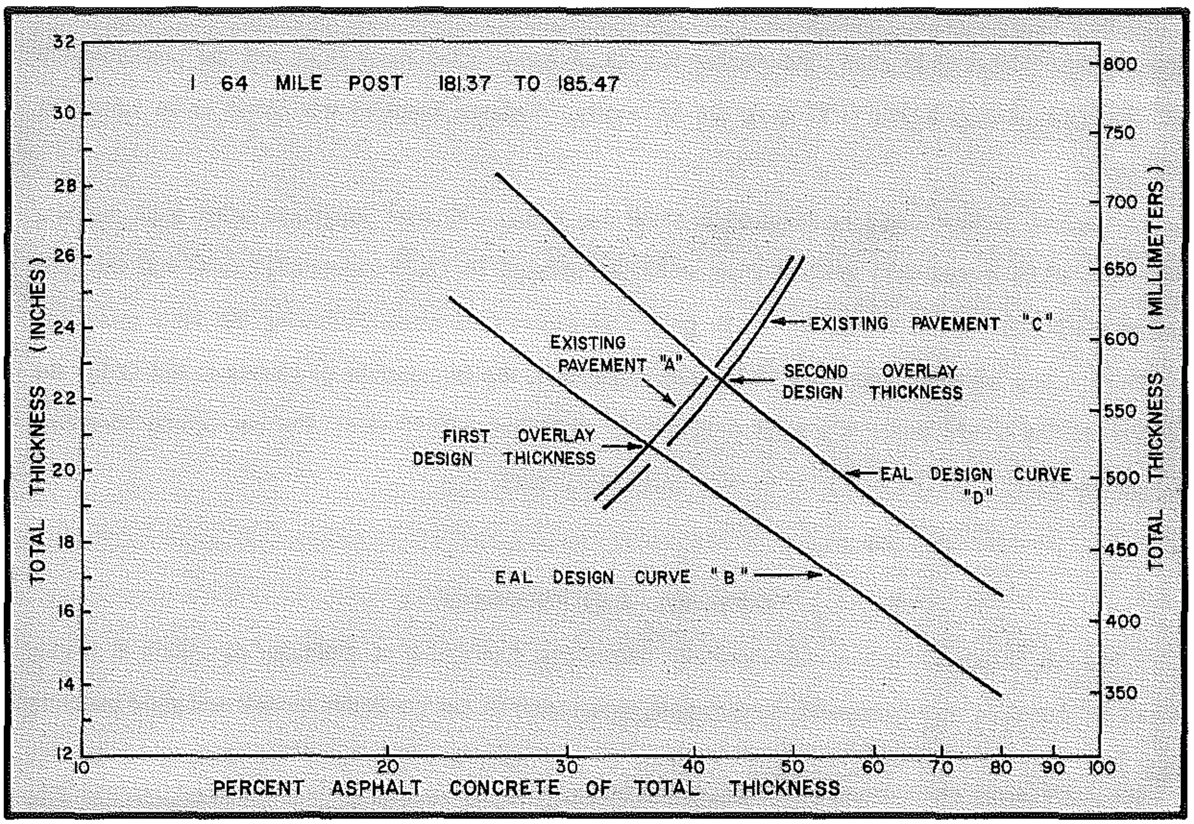


Figure 15. Percentage of Net Worth of Pavement after Disintegration Begins versus Percentage of Design Thickness.

Figure 16. Total Design Thickness versus Percentage of the Asphaltic Concrete Thickness of the Total Thickness.



The original design thickness for the DGA base was reduced according to the results of Figures 6b and 15. Continuing with I 64, Boyd County, as an example, the roughness index for 1977 was estimated to be 590, which corresponded to a serviceability index of 2.88 and a net worth of 80 percent. Figure 15 indicated the original 7.5 inches (190 mm) of AC was worth only 51 percent (3.85 inches (100 mm)) and the 15 inches (380 mm) of DGA was worth only 89 percent (13.25 inches (335 mm)).

Curve B (Figure 16) was constructed by determining the total thickness when the DGA proportion was a constant 13.25 inches (335 mm). Thus, for a 50-percent AC structure, the total thickness would be 26.50 inches (675 mm) and, for a 33-percent AC structure, would be 18.9 inches (480 mm), etc. The required total thickness was determined at the intersection of Curves A and B and, for this example, equaled 20.6 inches (525 mm). Subtracting the 13.25 inches (335 mm) DGA and the present-worth AC thickness of 3.85 inches (100 mm), the AC overlay thickness required was found to be 3.5 inches (90 mm).

A second overlay would be required in 1990 due to the estimated roughness index again reaching 460. The 13.25 inches (335 mm) of DGA would be reduced to 12.75 inches (325 mm) and the 7.35 inches (190 mm) of AC would be worth only 5.85 inches (150 mm). Following the same procedure as above, using the 1990 estimated 18-kip (80-kN) EAL's, the intersection of Curves C and D for the second overlay would result in a total thickness of 22.35 inches (570 mm). The net overlay thickness would be 22.35 - (12.75 + 5.85), or 3.75 inches (95 mm).

CONTROL AND FEEDBACK

Whereas feedback enables refinement of trends and fine-tuning of modeling equations, it is only when inputed field data (pavement condition measurements) fit the predictive models and yield matching schedules that automation is achieved. A misfit or mismatch would indicate the data to be in error -- or perhaps indicate premature deterioration of a pavement. An unrecognized or unresolved mismatch might induce the designer to revise the structural model to compensate for premature deterioration caused by influences assumed to be but really not related to live load and(or) to overdesign in an effort to minimize rutting. Truly, an eyeball inspection of pavement condition cannot be avoided. There are precautions to be taken and symptoms to beware of. Some roadway design features may, in reality, constitute a direct misfit insofar as the welfare of the pavement is concerned or else may induce unwanted side-effects. Furthermore, it should be mentioned, at least casually, that the controller must be disciplined and constrained by experience and extraordinary insights.

Whereas planners may fail at the outset to accurately forecast traffic and growth, or characteristics of vehicles, or loads, or traffic composition, or lane usage, certain alarm systems and control points should be built in. Perhaps not later than the half-life stage, as determined from annual estimates of EAL's accumulated in service or as determined from losses in serviceability index (from PSR), a cross-checking of trends should be ordered. Unless a mismatch between accumulated EAL's and loss in serviceability (or PSR) is found, an updating of estimated, remaining, service life may be made then and periodically thereafter. Decision points should be marked; lag time between decision points and overlaying or rehabilitation of the pavement must be counted; and the consequences of deferring action must be known. A strategy should be planned.

Figures 17 and 18 show deterioration mechanisms which were unanticipated in design but which emerged after the half-life stage. In Figure 17, frozen or misaligned dowel bars prevented the normal function of a contraction joint. The joint in the inside lane (in the background) remains locked; that joint in the outer lane is faulted (upthrust, creating a drop-off). Figure 18 shows a severe faulted crack following and between dowelled-but-locked (non-functioning) contraction joints. Successive or intermittent occurrences of this defect suffices to render a pavement unserviceable (as having a low PSR).

Figures 19 and 20 show similar deterioration at a joint advancing in both lanes and near-horizontal splitting of the slabs. The pavement as shown in Figure 19 has been sawed preparatory to lifting the section for examination *(18)*.

Figures 21 and 22 show progressive deterioration arising from severe temperature compression in a PCC pavement designed with sawed (reduced-section-type) contraction joints only (i.e., expansion joints were not provided). The interval between upheavals (small blowups) depends greatly on the strength of the concrete and the amount and regularity of deterioration at each joint. The greater interval ranges between two-tenths mile and a mile (0.3 kilometer and 1.6 kilometers). Minor deterioration is evident in the foreground of Figure 21; the blowup type is in the middleground. In Figure 22, the deterioration progressed from so-called D-cracking *(18)*.

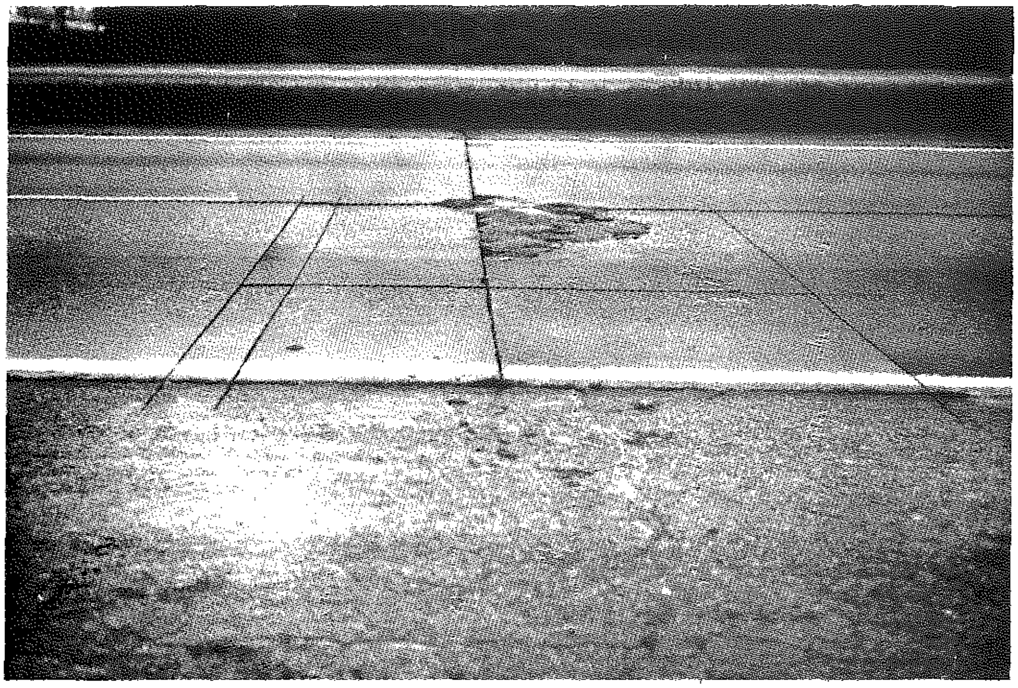
Figure 23 shows an explosive-type blowup which was triggered by temperature and those conditions known to be associated with weather and time of day. However, this pavement was unusually beset with or troubled in this way about once per mile (1.6 kilometers) before reaching its half-life age. Relief joints were cut (with some difficulty) and filled. An accompanying symptom is shown in Figure 24 (after the pattern became fully revealed). The early symptoms led to investigation and the discovery of expansive ledges of limestone in the aggregate quarry *(19)*.

Figure 25 illustrates the now-common problem of wear and rutting caused by studded tires running on PCC-type pavements. Because of slipperiness and channelized flow of water in the ruts, this section

Figure 17. Faulting of a Sawed, Doweled, Contraction Joint; Outer Lane.

Figure 18. Severe Faulting of a Crack Midway between Non-Functioning Contraction Joints.

Figure 19. Minor Humping at a Contraction Joint; Sawed in Preparation for Removal and Inspection.



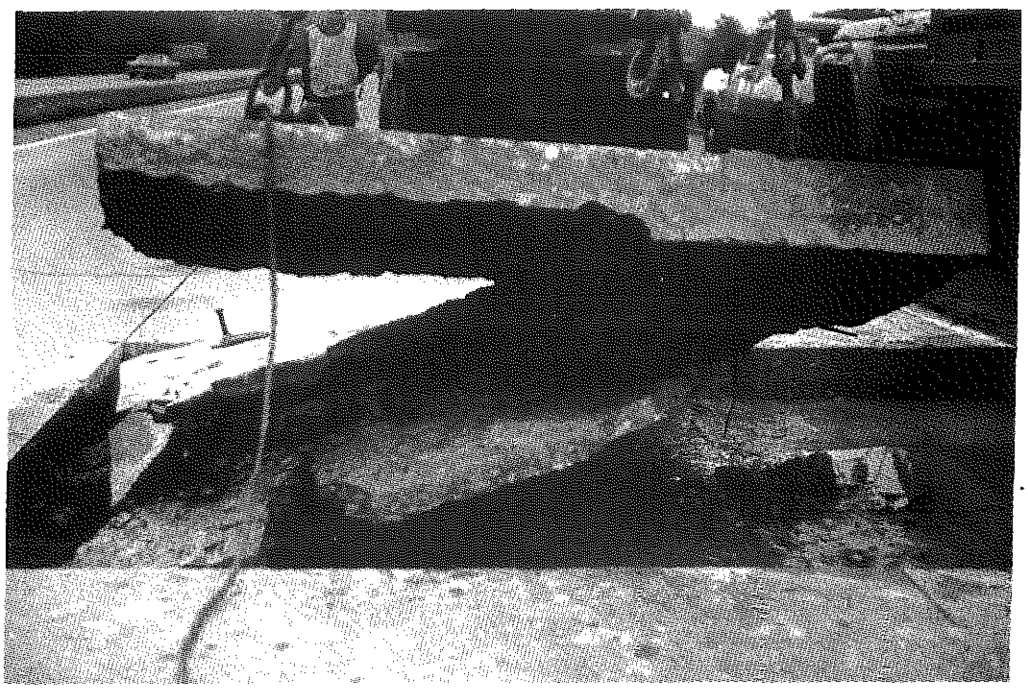


Figure 20. Exposed Joint and Surrounding Concrete; Same Site as Shown in Figure 19.

Figure 21. Progressive Deterioration, Arising from Temperature Compression; Sawed, Doweled, Contraction Joints.

Figure 22.   Advanced Stage of So-Called D-Cracking.

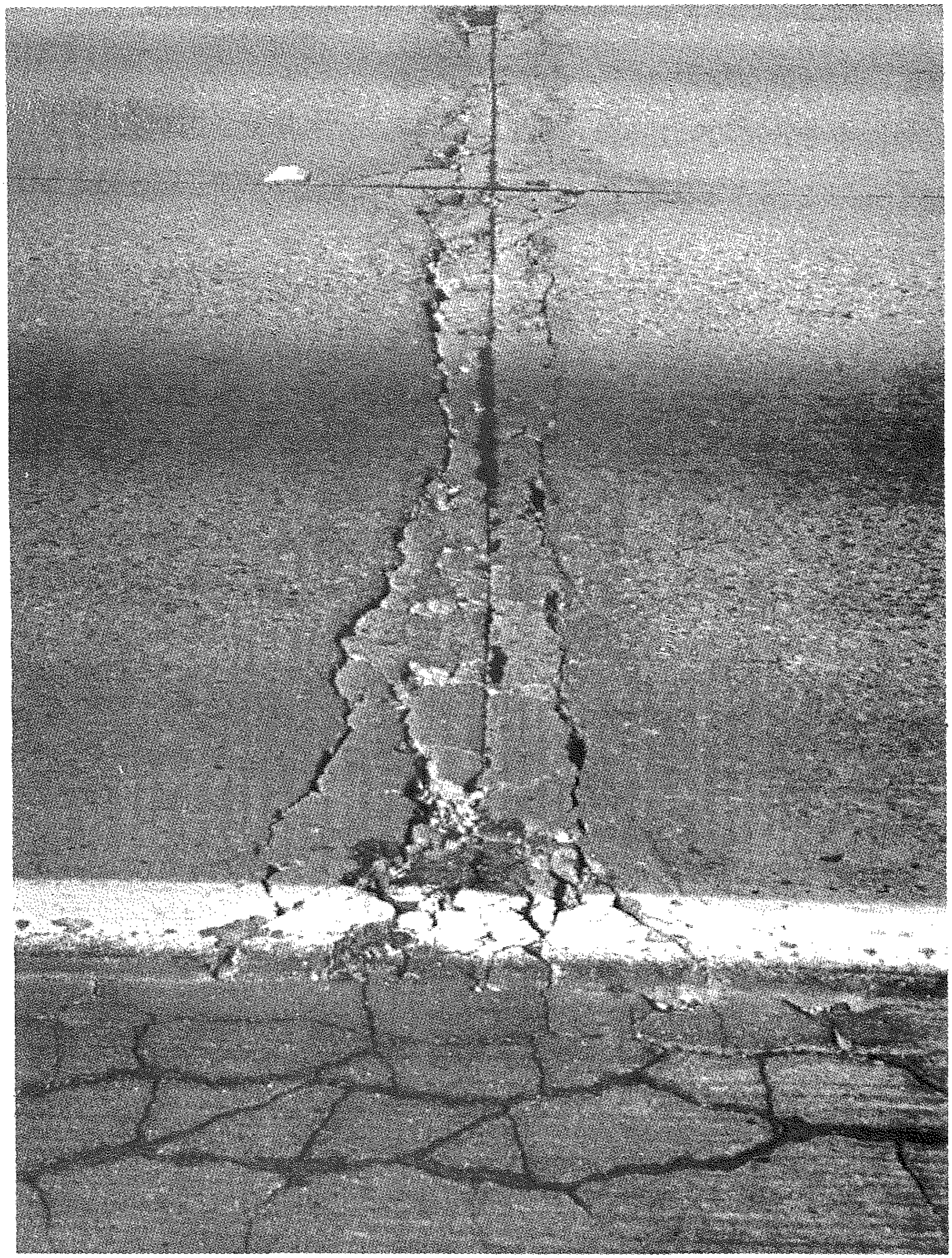


Figure 23. Explosive-Type Blowup, Triggered by Temperature Expansion; Attributed to Expansive Aggregates.

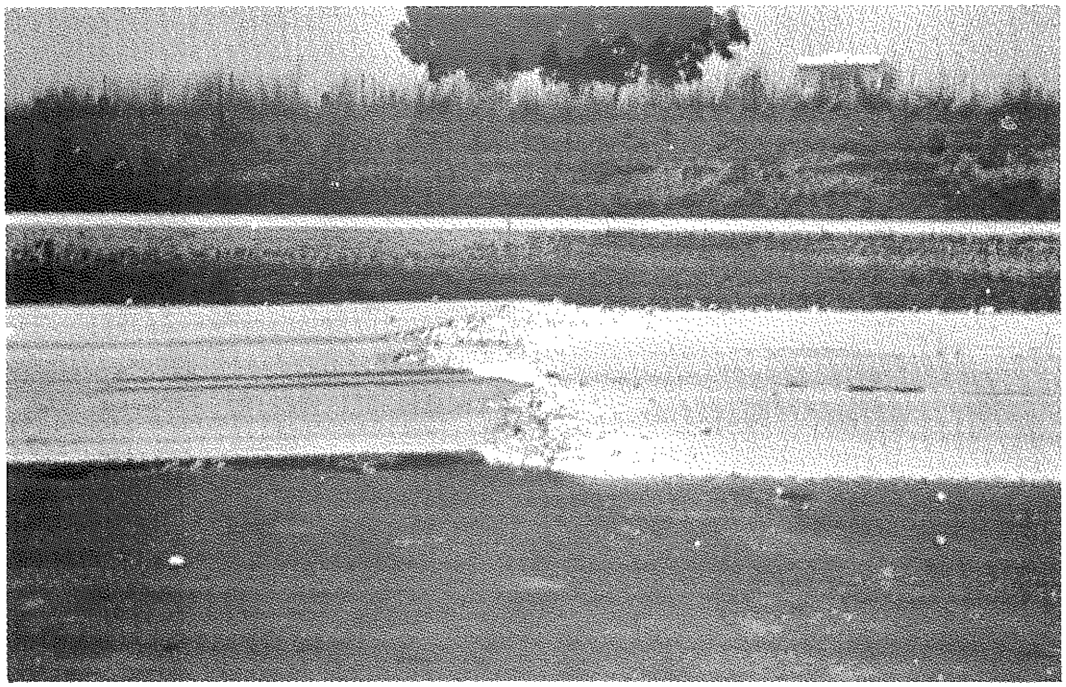


Figure 24. Surface Cracking, Symptomatic of Expansive Aggregate.

Figure 25. Wear and Rutting, Attributed to Studded Tires.

of pavement was overlaid (See Appendix, I 75, Covington, MP 190, approximately). A thin leveling course of asphaltic concrete and an open-graded friction course were applied. Traffic was maintained. The surface course raveled -- but remains unrepaired (a deferred action).

Figure 26 shows a typical but unwanted attribute of CRC-type pavements. Whereas some authorities argue that the cracking (close-spaced or otherwise) is of little consequence in the performance of the pavements, some argue to the contrary *(20)*. Figure 27 shows an explosive-type upheaval in a CRC pavement. There, too, the recognized conditions of weather and temperature trigger the action. There, specifically, a partial width patch had been made previously; and, in effect, the expansive force was borne on a reduced cross section.

Figure 28 shows a section of Interstate 75 which was overlaid (eligible for extension of service life to 1984) with bituminous concrete; traffic was maintained; bleeding and slickness developed; it was overlaid again with a sand-asphalt -- which proved to be unstable and scaled in the manner shown. This is a typical failing of low-stability, lean sand-asphalts and is traceable through the 1920's and 1930's and through the long history of Kentucky Rock Asphalt. The portion of the road shown was surfaced again (lastly) with bituminous concrete.

Figure 29 illustrates an unfortunate type of cracking in a relatively low-volume section of Interstate 64. Opinions differed as to the cause. The core in Figure 30 shows the typically shallow depth of the cracks (in surface course only). Roller marks persisted and were visible on the surface until it was overlaid. The possibility remained that the cracks had been induced by the compaction equipment (roller) at the time of construction and merely became evident after weathering and surface erosion. Figure 31 further illustrates an unfortunate tendency for glazing to occur in wheelpaths following bituminous resurfacing when traffic is maintained (no curing or setting time provided).

Figure 32 serves as a reminder of the inherent tendency for bituminous pavements to rut in some proportion to wheel loadings, number, and speed of vehicle. Considerations from the standpoint of safety or technological advances may alter decision and action patterns. A case in point is illustrated here and in Figure 33. The latter shows the texture obtained by planning with the CMI Roto-Mill. The ruts have been eliminated, but the final strategy has not been planned.

From a structural point of view, this operation could be considered counterproductive or as decreasing the asset value of the pavement. The economic feasibility of planning depends, of course, on a situation analysis and the strategies planned and applied. A rather current question concerns the feasibility of planning PCC pavements exhibiting faulting at joints and cracks such as shown in Figures 17 and 18 -- but in some states associated with sawed-type joints but not doweled.

Figure 26. Typical Cracking in CRC-Type Pavement.

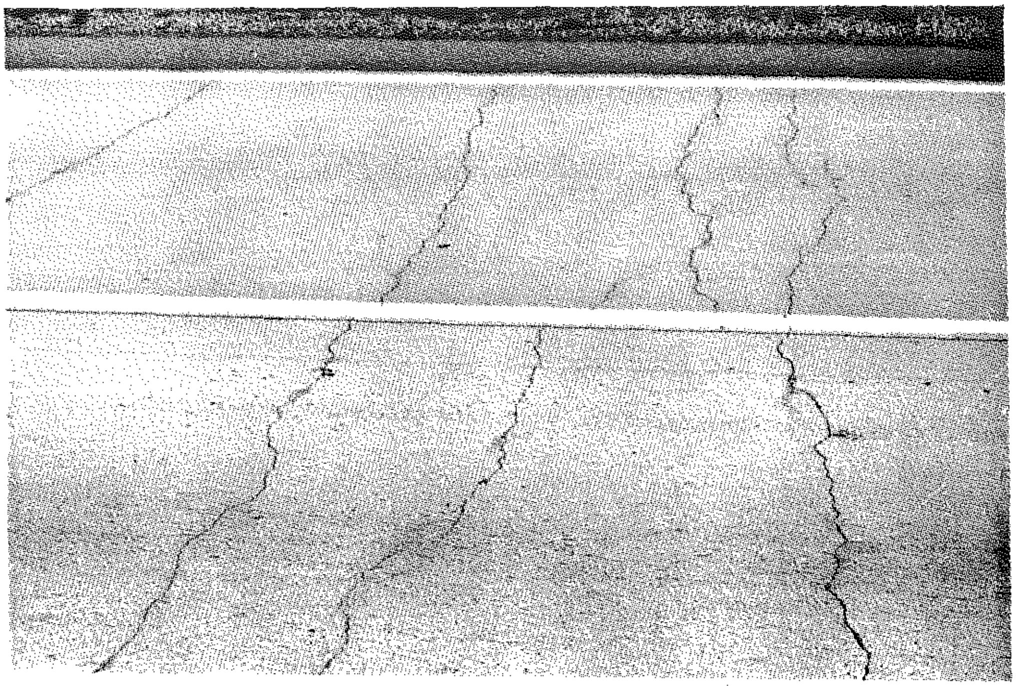


Figure 27. Explosive Upheaval in a CRC-Type Pavement.

Figure 28. Scaling of an Unstable Sand-Asphalt.

Figure 29. Cracking in an Asphaltic Concrete Surface Course.

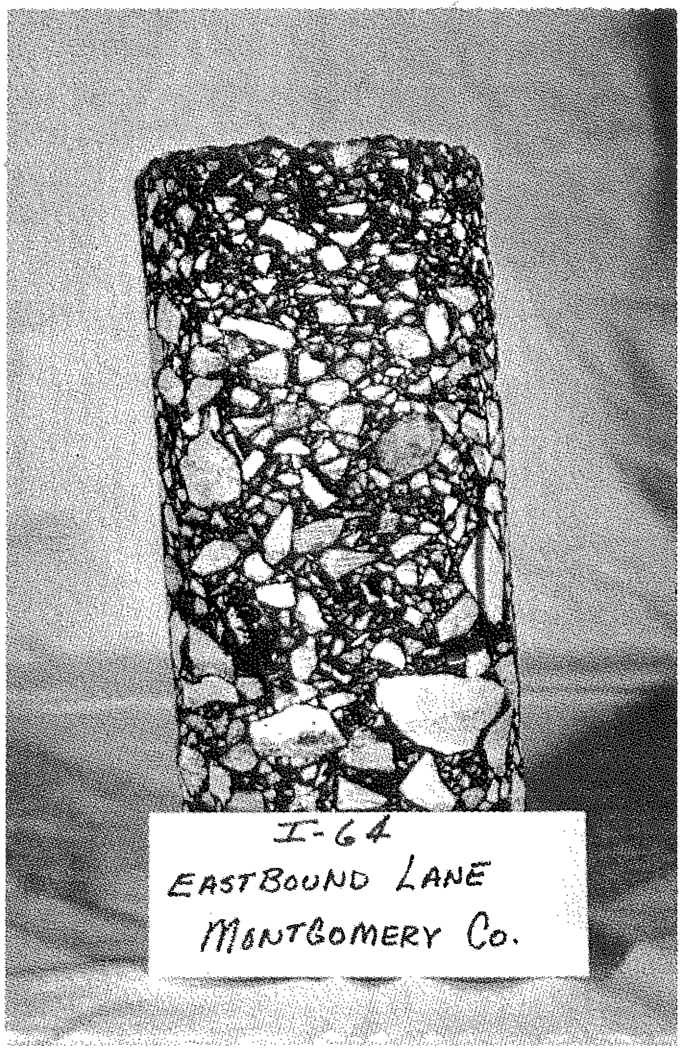


Figure 30. Core Showing Shallow Cracking; Surface Course.

Figure 31. New Asphaltic Concrete Surface Constructed under Traffic; Tendency for Glazing and Apparent Bleeding Shown in Background.

Figure 32. Tendency toward Rutting in Asphaltic Concrete Pavements Carrying Heavy Traffic.

Figure 33. Texture Produced by Planing Rutted Asphaltic Concrete Pavement; CMI Roto-Mill.

REFERENCES

1. Southgate, H. F.; Deen, R. C.; Havens, J. H.; and Drake, W. B.; *Kentucky Research: A Flexible Pavement Design and Management System,* **Proceedings**, Fourth International Conference on the Structural Design of Asphalt Pavements, Vol 1, 1977.
2. Newberry, D. C. Jr.; Mayes, J. G.; and Havens, J. H.; *Highway Pavement Maintenance Costs and Pavement Type Selection,* Division of Research, Kentucky Bureau of Highways, 1974.
3. Sedmak, A.; and Domaschuk, L.; **Theoretical Analysis of Structural Behavior of Road Test Flexible Pavements,** NCHRP Report 122, Highway Research Board, 1971.
4. Winfrey, R.; and Zellner, C.; **Summary** and **Evaluation of Economic Consequences of Highway Improvements,** NCHRP Report 122, Highway Research Board 1971.
5. Van Til, C. J.; McCullough, B. F.; Vallerga, B. A.; and Hicks, R. G.; **Evaluation of AASHO Interim Guides for Design of Pavement Structures,** NCHRP Report 128, Highway Research Board, 1972.
6. Hudson, W. R.; and McCullough, B. F.; *Flexible Pavement Design and Management: Systems Formulations,* NCHRP Report 139, Highway Research Board, 1973.
7. Lytton, R. L.; McFarland, W. F.; and Schafer, D. L.; **Flexible Pavement Design and Management: Systems Approach Implementation,** NCHRP Report 160, Transportation Board, 1975.
8. Drake, W. B.; and Havens, J. H.; *Kentucky Flexible Pavement Deisgn Studies,* **Bulletin No. 52,** Engineering Experiment Station, University of Kentucky, 1959; also **Bulletin 233,** Highway Research Board, 1959.
9. Pigman, J. G.; Agent, K. R.; Mayes, J. G.; and Zegeer, C. V.; *Optimal Highway Safety Improvement Investments by Dynamic Programming,* Division of Research, Kentucky Bureau of Highways, 1974 (also in **Record** 585, Transportation Research Board, 1976); also see Zegeer, C. V.; Agent, K. R.; and Rizenbergs, R. L.; *Use of Dynamic Programming in the Selection of Projects for Resurfacing,* pending, Division of Research, Kentucky Bureau of Highways.
10. 1976 Federal-Aid Highway Act, Section 150(b); Resurfacing, Restoration and Rehabilitation Study (OMB No. 004-S76034), FHWA Notice N5020.10, January 27, 1977; also see Southgate, H. F.; Newberry, D. C.; Deen, R. C.; and Havens, J. H.; *Resurfacing, Restoration, and Rehabilitation of Interstate Highways: Criteria and Logic Used To Determine January 3, 1977, Needs and Estimates of Costs,* Division of Research, Kentucky Bureau of Highways, July 1977.
11. Lynch, R. L.; and Hamby, G. N.; *Lateral Distribution of Traffic on a Four-Lane and Six-Lane Section of I 75 South of Covington,* Division of Research, Kentucky Department of Highways, June 1969.

12. **Truck Weight Vehicle Classification Study**, Kentucky Department of Transportation, Division of Statewide Transportation Systems Planning, Annual Reports, 1959-1975.
13. Pigman, J. G.; and Mayes, J. G.; *Characteristics of Traffic Streams on Rural, Multilane Highways,* Division of Research, Kentucky Bureau of Highways, April 1976.
14. **AASHTO Interhn Guide for Design of Pavement Structures**, 1974.
15. Rizenbergs, R. L.; Burchett, J. L.; and Davis, L. E.; *Pavement Roughness: Measurement and Evaluation,* Division of Research, Kentucky Department of Highways, December 1971.
16. **Thickness Design for Concrete Pavements**, Portland Cement Association, **Bulletin HB 35, 1966.**
17. Southgate, H. F.; and Deen, R. C.; *Temperature Distribution in Asphaltic Concrete Pavements,* **Record 549**, Transportation Research Board, 1975.
18. Havens, J. H.; *The D-cracking Phenomenon: A Case Study for Pavement Rehabilitation,* Division of Research, Kentucky Bureau of Highways, April 1976.
19. Havens, J. H.; and Rahal, A. S.; *Expansive Limestone Aggregate in a Concrete Pavement,* Division of Research, Kentucky Department of Highways, April 1972.
20. Havens, J. H.; Deen, R. C.; Rahal, A. S.; and Azevedo, W. V.; *Cracking in Continuously Reinforced Concrete Pavements,* Division of Research, Kentucky Bureau of Highways, October 1977.

APPENDIX

KENTUCKY

Resurfacing, Reconstruction, Rehabilitation

(R-R-R)

Kentucky's annual construction program in the last 25 years has increased approximately ten fold from $20 million - $30 million to $200 million - $300 million, and this increased road building program has solved many resurfacing problems. It has also created future maintenance problems as more mileage has been added to the system; therefore, more strain has been placed on the maintenance budget. With the limited funds assigned to maintenance, it has not been difficult to establish resurfacing and reconstruction priorities. During this same 25-year period, allowable gross weights have doubled from 40,000 to 80,000 pounds (18 to 36 Mg). However, the most significant factor in the design of roads, including pavements, has been the inability to forecast the traffic variables. Traffic life has frequently been reached in a 1/3 to 1/2 of the 20-year design life.

The maintenance engineer has had needs exceeding available funds in large multiples. A parable comes to mind of today's housewife who goes to the grocery with $20 to buy weekly food for a family of five. While it is almost an impossible task, priorities must be developed to spend the available funds wisely.

Interstate cost estimated for the R-R-R program in Kentucky between now and 1990 exceeds $350 million based on 1975 costs. However, the amount allotted for the last fiscal year was $2.9 million; and another, like amount is anticipated for next year. Of course, other states have experienced a similar allottment. With these needs and these allocations, it is not difficult to determine the highest priority. Historically this has been the case over the years; there just has not been enough money to do the job that needs to be done. The most significant aspect of this problem is what has been done over the years in resurfacing and rehabilitation, and following are projects which are examples of that effort.

First, three resurfacing projects done on the Interstate System are described and are representative of conventional resurfacing. Second, two reconstruction projects are described; and they represent intermediate efforts to correct a significant deficiency and not bring the road to any standard level of service. Third, three projects reconstructed to current standards for a 20-year service life are described. Fourth, are the R-R-R projects which are more representative of today's resurfacing and reconstruction program. Fifth is an example of a reconstruction safety project, and sixth and last will be a project that started out to salvage the old pavement but ended up with a complete new template. In addition to typical pavement sections for the various projects, recent photographs attached are intended to reveal the typical condition of the pavements.

1. Jefferson County; I 264-1(38)8;

I 264; Watterson Expressway;

From near US 60W to US 60 E;

1968; 11.0 miles (17.7 kilometers).

This portion of the Watterson, exclusive of the I-64 interchange, was originally constructed with flexible pavement with federal non-interstate funds and then taken into the Interstate System. It was eligible for interstate resurfacing because it had not been designed to interstate criteria. A minimum 1-inch (25-mm) binder and 1-inch (25-mm) surface were applied as Stage-1 resurfacing and have performed very adequately for some 9 years. (See typical section, identified as No. 1.)

2. Clark-Montgomery Counties; I 64-5(29)89 & I 64-5(30)101;

I 64;

From near Fayette County Line to US 60 Interchange, east of Mt. Sterling;

1972; 11.846 = 23.114 miles (37.190 kilometers).

The eligibility for interstate funding for the resurfacing of these two projects was the result of changing design criteria to 20-year traffic as opposed to 1975 traffic. The first portion from the Fayette County Line to the Mountain Parkway received 2 1/2 inches (65 mm) of bituminous concrete surface in two courses (1 1/2 inches (38 mm) + 1 inch (25 mm)). The inside shoulders were extended; however, the outside shoulders were wedged to conserve material and preserve the guardrail height. The second portion from the Mountain Parkway to the US-60 Interchange east of Mt. Sterling was eligible for only 2 inches (51 mm); therefore, two 1-inch (25-mm) courses were used. Shoulders were treated in a like manner, and the same contractor paved both projects. (See typical sections, identified as Nos. 2 and 2A.)

3. Fayette-Madison Counties; I 75-4(32)90 & I 75-3(36)87;

I 75

KY 876 to Grimes Mill Road;

1971; 13.1 miles (21.078 kilometers).

This project was eligible for interstate funds the same as the previously described project and is, therefore, considered Stage-2 paving. Two 1-inch (25-mm) courses of surface were specified; the shoulders, however, could be paved either in one or two lifts to permit them to be paved as a part of the driving lane or separately. Resurfacing was accomplished in conjunction with minor safety improvements such as minor drainage and grading, guardrail, sign improvements, etc. Relatively thin overlays described in these projects has been the rule as opposed to the exception. (See typical section, identified as No. 3.)

Following are two examples of reconstruction projects which are considered improvements; however, they are not to the ultimate design standard. Administrative decisions are frequently made in the interest of safety to upgrade a facility to minimum standards because of a lack of funds.

4. Fayette-Woodford Counties; SP 34-164 & SP 150-95;

US 60;

Lexington to Versailles;

1965; 7.202 miles (11.588 kilometers).

This project was originally constructed in the late 1930's as a four-lane (9.25 feet (2.8 m) each) facility with a 4-foot (1.2-m) grass median and 7-foot (2.1-m) shoulders. According to standards of that time (1965) the roadways were deficient in most every way; however, as an interim measure, it was decided to widen the pavement to four, 12-foot (3.7-m) lanes by utilizing the available shoulder. It was a good decision as it gave the motorist a safer feeling in passing, especially while passing wide trucks. Though the speed limit is 50 mph (22 m/s), there are still many accidents and fatalities. It is thought that the accident rates would have been much higher without the pavement widening. Overlay thicknesses were held to a minimum (1 1/4 inches (32 mm) of binder and 1 inch (25 mm) of surface); and, due to the truck traffic, reflective cracking began to show early. This road is now in the design phase to bring it to current, 20-year standards for median, pavement, shoulder, ditches, alignment, and grade as well as superelevation. (See typical section, identified as No. 4.)

5. Ballard-Carlisle-Hickman-Fulton Counties; US 51;

SP 53-129-4, SP GR12-66, SP 4-61-4, SP GR 57-62;

Wickliff to Fulton;

1962-1966; 27.7 miles (44.6 kilometers).

This route runs north-south through the most western part of the state from Wickliff to Fulton, and complete relocation was considered at one time. All factors taken into account including other available routes, it was decided to upgrade the existing facility as much as possible and stay on the existing right of way. For the most part, this meant widening the pavement on each side, resurfacing, and bridge widening. The existing 9-6-9 concrete pavement was constructed in 1930 to an 18-foot (5.5-m) width in the rural areas, and curb-and-gutter sections were used through the small towns. Two feet (0.6 m) of the existing shoulders were sacrificed for pavement widening, making 11-foot (3.4-m) lanes. This effort was accomplished in a series of reconstruction projects and completed under traffic. Some widening of bridges was done with maintenance forces. Relatively thin bituminous concrete overlays were again used; and reflective cracking was anticipated. This type of reconstruction effort is justified in that it provides a much safer operating facility over a period of time. Although a significant amount of truck traffic is operating over the road, reflective cracking has proven to be very insignificant in the 10 to 15 years of service. This is attributed in the most part to the excellent subgrade of native granular materials. (See typical sections, identified as Nos. 5 and 5A.)

6. Daviess County; F 105(12);

US 60; Owensboro to Maceo;

1965; 5.639 miles (9.073 kilometers).

Like the US-51 and US-60 projects previously described, this is an old concrete pavement of the 9-6-9 variety constructed in 1930 to an 18-foot (5.5-m) width. Relatively good alignment and grade exist due to the terrain. In 1965, this road was brought to current, 20-year, projected, traffic standards and constructed with federal, matching funds. A 24-foot (7.3-m) pavement, 10-foot (3.0-m) shoulders, and 6-foot (1.8-m) ditches were necessary to meet this criteria. Approximately 4 miles (6.4 kilometers) of the 5.6 miles (9.0 kilometers) total is conventional concrete base widening and resurfacing. All the widening was done on one side; and the crown was shifted with bituminous concrete binder. Short sections of low verticals were corrected with bituminous concrete and with DGA up to 12 inches (0.3 m) thick. DGA base was used up to 18 inches (0.4 m) in depth over the old pavement. When the depth of DGA would have exceeded 18 inches (0.4 m), which occurred for 6,000 feet (1.83 kilometers), a completely new paving template was employed; and the old pavement was broken and left in place. The varying sections make the project look difficult to construct; however, it was not difficult; and the pavement has performed exceptionally well. Again, minimum bituminous overlays were employed (See typical sections, identified as Nos. 6, 6A, 6B, 6C, and 6D.)

7. Boyd County; APD 537(51)(57)(58);

US 23;

From Ashland to Catlettsburg;

1972-1973; 4.946 miles (7.958 kilometers).

The existing concrete pavement became eligible for salvage inasmuch as large quantities fit the alignment and grade of the new proposed, four-lane construction and was in relatively satisfactory condition. Five, typical sections are shown as representative of the variables peculiar to this work. Section 7, 7A, and 7B were developed as the grade was raised over the existing pavement. Section 7A is for a minimum depth; Section 7 is for a significant length; and 7B accommodates a large grade change or dip in the old road. Section 7C was used where the survey hit the middle of the old road; and Section 7D was used where the old road was undercut and a complete new template was needed. Maintaining traffic on the old road was a big factor in deciding to salvage. (See typical sections, identified as Nos. 7, 7A, 7B, 7C, and 7D.)

8. Jefferson County; I 65-6(34)130;

I 65; North South Expressway;

Watterson Expressway to Woodbine;

1970; 3.392 miles (5.458 kilometers).

The original project was constructed in 1955; it had four lanes, a 30-foot (9.1-m), raised grass median; and 8 feet (2.4 m) of a 10-foot (3.0-m), outside shoulder was paved. The reconstruction project was in 1970 and consisted of adding two lanes in the median, a concrete barrier wall in the median, bridge widening on the inside, and Stage-1 of a bituminous concrete overlay. The bituminous concrete overlays were eligible for federal interstate funds on the same basis as described previously for the three resurfacing projects -- which was a change in federal criteria. Two-lane traffic in each direction was maintained with a minimum of inconvenience to the public. Bituminous paving was done at night at the option of the contractor. Outside widening of bridges is staged to be done at a later date and will comply with current standards. The original, four-lane construction was designed to allow for future, six-lane expansion in that the bridges had temporary parapets and bridge piling was originally installed. This project is shown as an excellent example of anticipated, future construction allowed for fully in the best of fashion in the original work. Another resurfacing is currently scheduled in the 1978-1979 construction season. (See typical section, identified as No. 8.)

9. Boone County; I 75-7(53)(54)(55)(56);

I 75;

From I 71 to US 42;

1978; 6.25 miles (10.06 kilometers).

This is the most significant reconstruction project as relates to pavement that has been let to contract on the interstate system in Kentucky. The federal R-R-R program prompted this report for only 6 miles (9.6 kilometers) of an ultimate 16-mile (25.7-kilometer) upgrading. The original pavement was constructed of 10 inches (0.25 m) of portland cement concrete, with mesh and dowels, built in 1960; the inside lanes were added in 1965. The reason this project is considered in the reconstruction category is the inside shoulder is to be widened from 6 feet (1.8 m) to 10 feet (3.0 m) to accommodate a disabled vehicle, which is a criteria change from the original design. The problem of maintaining overhead bridge clearance was intentionally avoided as the section was selected outside the limits of affected structures. Various alternatives are being studied for sections where bridges are involved; these alternatives include (1) raising overhead structures, (2) removing pavement and lowering grades, (3) reducing overlay thicknesses within affected areas of influence. Eight inches (0.20 m) of bituminous concrete overlay is being constructed throughout the six lanes and shoulders plus an open-graded, plant-mixed seal on the driving lanes; this is the heaviest resurfacing project of this magnitude ever done in Kentucky. Four-inch (102-mm), perforated underdrain pipe was included along each pavement edge in an effort to improve drainage around the pavement. No.-57 stone was substituted in lieu of a natural sand backfill because natural sand was seeping into the pipe causing settlement. Plastic, polyethylene, corrugated pipe is being used at the contractor's option because it is less expensive than other available alternatives.

The contractor was given the option of raising the elevation of the existing guardrail by either removing and replacing or by utilizing an 8-inch (0.20-m), extended, modified, steel, off-set block. It appears likely that the contractor will choose to use the off-set blocks -- which will be a first-of-its-kind. Surely, consideration will be given to including this in the national standards.

Proof-rolling with a 50-ton (45-Mg) roller was required in an effort to determine weakened joints in the concrete; D-cracking was widespread; and deterioration of the joints was far advanced. When weak joints are located, the slabs are broken into pieces with a 2-foot (0.6-m) maximum size. The project will be paved under traffic; the contractor will be required to maintain two lanes in each direction at all times; shoulder will be available for this purpose.

Thirty-foot (9.1-m) sections of temporary, concrete barrier wall are required on the approach ends of the bridge widening to protect both the workmen and the traffic during the widening of the structure. Paving on this project is anticipated to start this fall. (See typical sections, identified as Nos. 9 and 9A.)

10. Kenton County; I 75-8(25)176 C;

I 75;

From US 25 - US 42 Interchange to the Ohio River Bridge;

1969; 3.78 miles (6.08 kilometers).

In 1969, the median on the subject project was reconstructed by converting a raised, curbed, 20-foot (6.1-m), earth median with guardrails to a 20-foot (6.1-m), flush median with a concrete barrier wall down the center. Cross-median accidents were eliminated completely. Numerous breakouts have occurred in the eccentric section as a result of requiring only a 6-inch (152-mm) top width. Today, design would be more stable by making the section wider. This is an excellent application of a median, safety, reconstruction project. All work was accomplished under traffic; inside lane closure was permitted in short sections during work hours. Work was not permitted during heavy peak flows which occurred in the mornings and evenings and on weekends during ball games and on holidays. These conditions were spelled out in the proposal very adequately. A couple of maintenance cross-overs in the median were closed; this necessitated longer runs for the winter salt trucks; however, it was done in the interest of safety; and maintenance crews have adjusted accordingly. (See typical sections, identified as Nos. 10 and 10A.)

11. Hardin-Bullitt-Jefferson Counties;

TP 15-574-43L; TP 47-69-29L; and TP 56-688-30L;

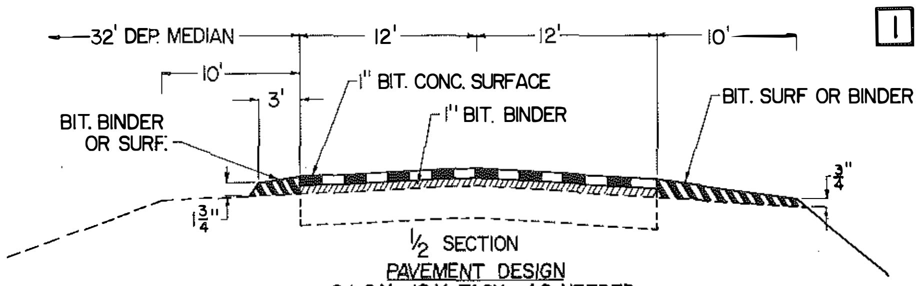
I 65;

From Elizabethtown to Louisville (Watterson Expressway);

Under design; 37.39 miles (60.16 kilometers).

This project was originally constructed as the first turnpike in Kentucky almost 20 years ago and exceeded all expectations for traffic volumes. It was taken into the interstate system some years ago and freed of toll last year. It is a four-lane facility with a 20-foot (6.1-m) earth median; the project is now in the design phase; it is scheduled for upgrading to a six-lane, 60-foot (18.3-m) depressed-median section. Full safety standards are applied to the rural portion; and the urban portions in Jefferson County are being designed for four lanes with dual roadways on each side with connectors at interchanges. It was conceived and the line and grade established anticipating salvaging of the old pavements; however, due to current standards and the condition of the old pavement, this concept was abandoned. It is now planned to construct new concrete pavement throughout the limits of the project, with no salvage of the old pavement. This project is mentioned here because every effort was made to utilize the old pavement; however, it became apparent that less than 25 percent of the new pavement would contain salvaged pavement in the rural portions and less than 15 percent in the urban sections. The value received for the money saved was very questionable; and a completely new pavement throughout was determined to be in the best public interest. (See typical sections, identified as Nos. 11 and 11A.)

Due primarily to increased traffic volume and weight, Kentucky can be expected to use heavier overlays on future resurfacing and reconstruction projects. Reflective cracking of rigid pavements and rutting of flexible pavements are still the problems to overcome in pavements. The objective is to maintain a safe, comfortable riding surface for the public.

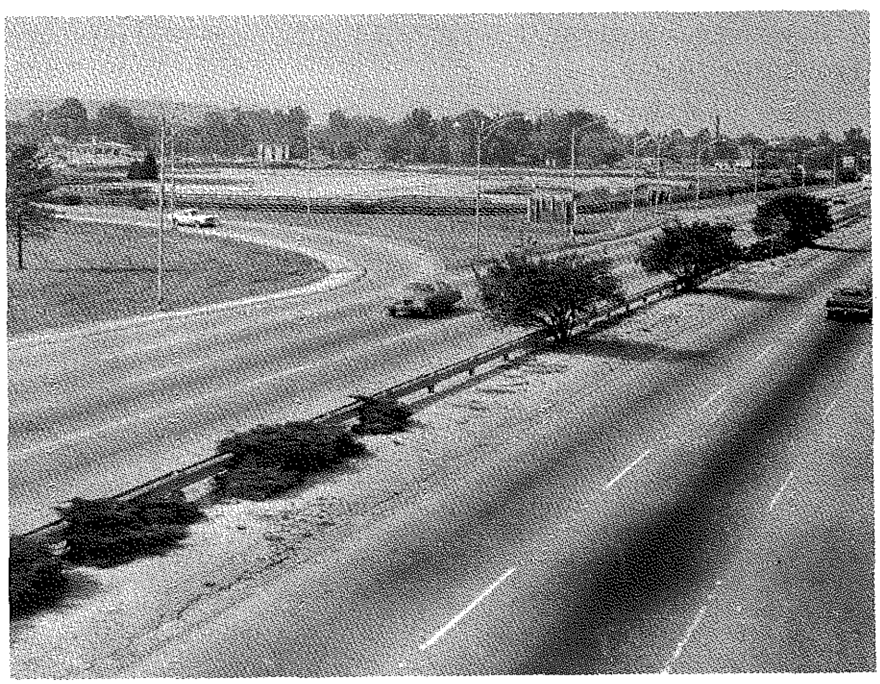


½ SECTION
PAVEMENT DESIGN
0.1 GAL./S.Y. TACK - AS NEEDED

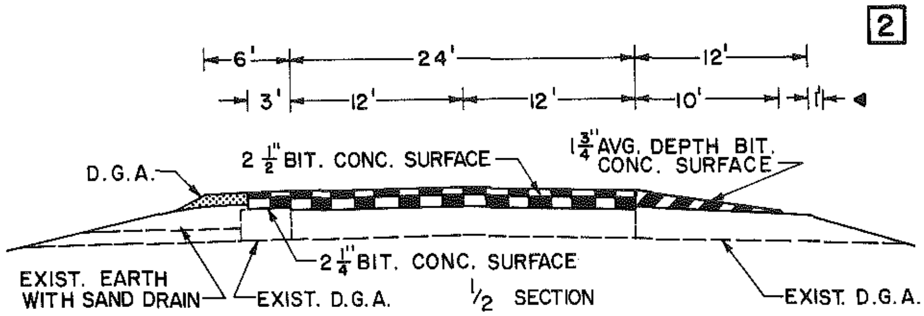
PAVEMENT
1000 TONS/MILE BIT. CONC.
FOR LEVELING & WEDGING
1" COMPT. BIT. CONC. BINDER
1" COMPT. BIT. CONC. SURFACE

SHOULDERS
0.35 GAL./S.Y. SEAL COAT
20 LBS./S.Y. CR. LIMESTONE
SHOULDER-INSIDE
$1\frac{3}{4}$" COMPT. BIT. CONC. SURFACE OR BINDER
SHOULDER - OUTSIDE
$1\frac{1}{4}$" AVG. COMPT. BIT. CONC.
SURFACE OR BINDER

I-264-I JEFFERSON CO. - WATTERSON EXPRESSWAY - FROM MANSLICK RD. TO U.S. 60 INTCHG.
(38) 8 EAST   11 MILES



Jefferson County; I 264; Watterson Expressway; From Near US 60W to US 60E; Resurfaced in 1968; Recent Photo.



PAVEMENT DESIGN

PAVEMENT

0.10 GAL. TACK /SQ. YD.
1,000 TONS / MILE BIT. CONC. SURFACE FOR LEVELING & WEDGING
$1\frac{1}{2}$" + 1" COMPT. BIT. CONC. SURFACE

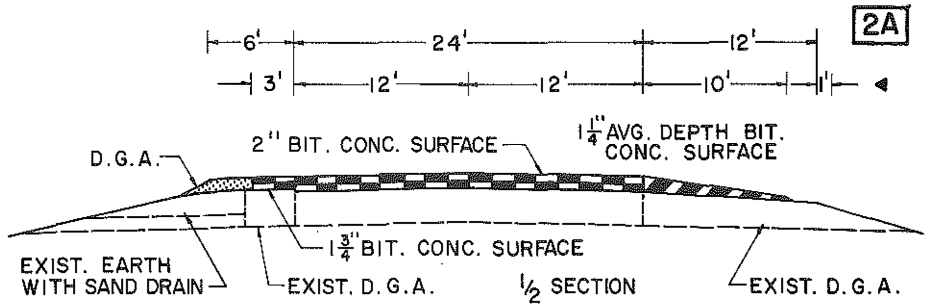
SHOULDER (INSIDE)

0.10 GAL. TACK / SQ. YD.
0.30 GAL. SEAL COAT / SQ. YD.
20 LBS. COMPT. CRUSHED AGGR., SIZE NO. 8 / SQ. YD.
$1\frac{1}{2}$" + $\frac{3}{4}$" COMPT. BIT. CONC. SURFACE

SHOULDER (OUTSIDE)

0.10 GAL. TACK / SQ. YD.
▶ 0.30 GAL. SEAL COAT / SQ.YD.
▶ 20 LBS. COMPT. CRUSHED AGGR. SIZE NO. 8 / SQ. YD.
$1\frac{3}{4}$" AVG. DEPTH BIT. CONC. SURFACE ($2\frac{1}{4}$" AT PAVEMENT EDGE AND $1\frac{1}{4}$" AT 10' POINT
▶ EXTEND 1' DOWN FILL OR DITCH SLOPE

I 64-5(29) 89 CLARK CO. 11.846 MILES
HALEY ROAD TO MOUNTAIN PARKWAY



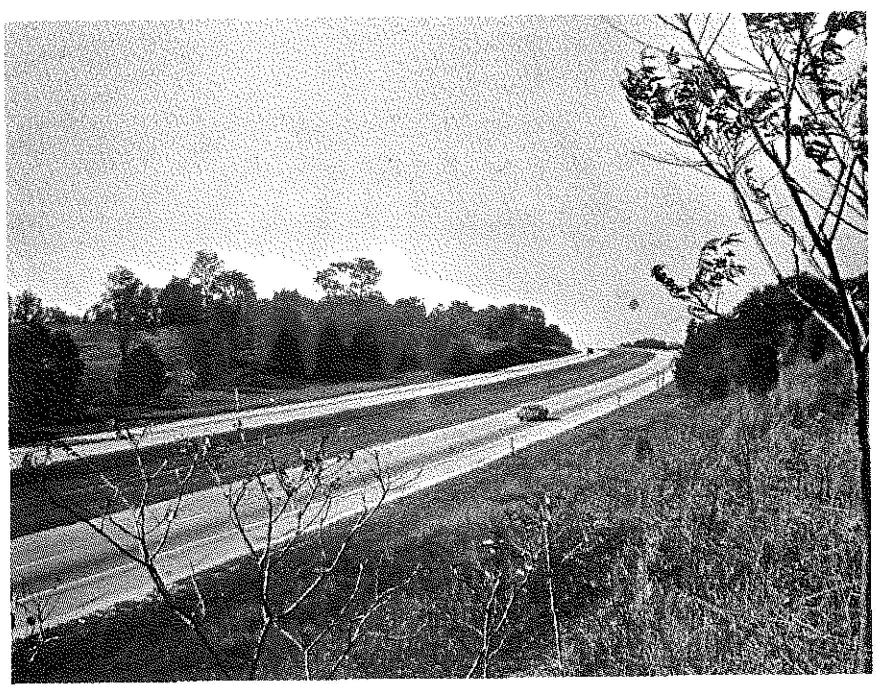
PAVEMENT DESIGN

PAVEMENT
0.10 GAL. TACK / SQ. YD.
1,000 TONS / MILE BIT. CONC. SURFACE FOR LEVELING & WEDGING
1" + 1" COMPT. BIT. CONC. SURFACE

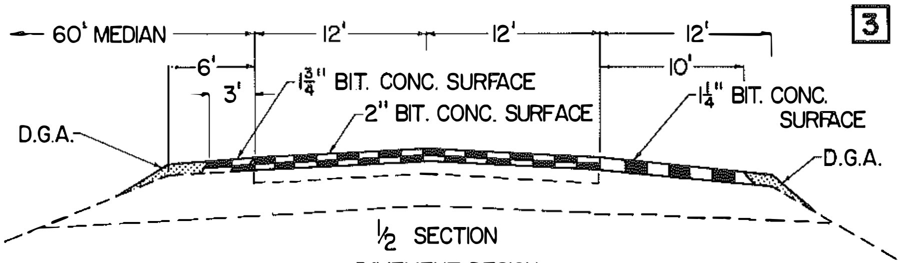
SHOULDER (INSIDE)
0.10 GAL. TACK / SQ. YD.
0.30 GAL. SEAL COAT / SQ. YD.
20 LBS. COMPT. CRUSHED AGGR. SIZE NO. 8 / SQ. YD.
$1" + \frac{3}{4}"$ COMPT. BIT. CONC. SURFACE

SHOULDER (OUTSIDE)
0.10 GAL. TACK / SQ. YD.
▶ 0.30 GAL. SEAL COAT / SQ. YD.
▶ 20 LBS. COMPT. CRUSHED AGGR. SIZE NO. 8 / SQ. YD.
$1\frac{1}{4}$" AVG. DEPTH BIT. CONC. SURFACE ($1\frac{3}{4}$" AT PAVEMENT EDGE AND $\frac{3}{4}$" AT 10' POINT
▶ EXTEND 1' DOWN FILL OR DITCH SLOPE

I 64 – 5 (30) 101 CLARK – MONTGOMERY COUNTIES
MOUNTAIN PARKWAY TO U.S. 60 11.27 MILES



Clark-Montgomery Counties; I 64; From Near Fayette County Line to US-60 Interchange East of Mt. Sterling; Resurfaced in 1972; Recent Photo.

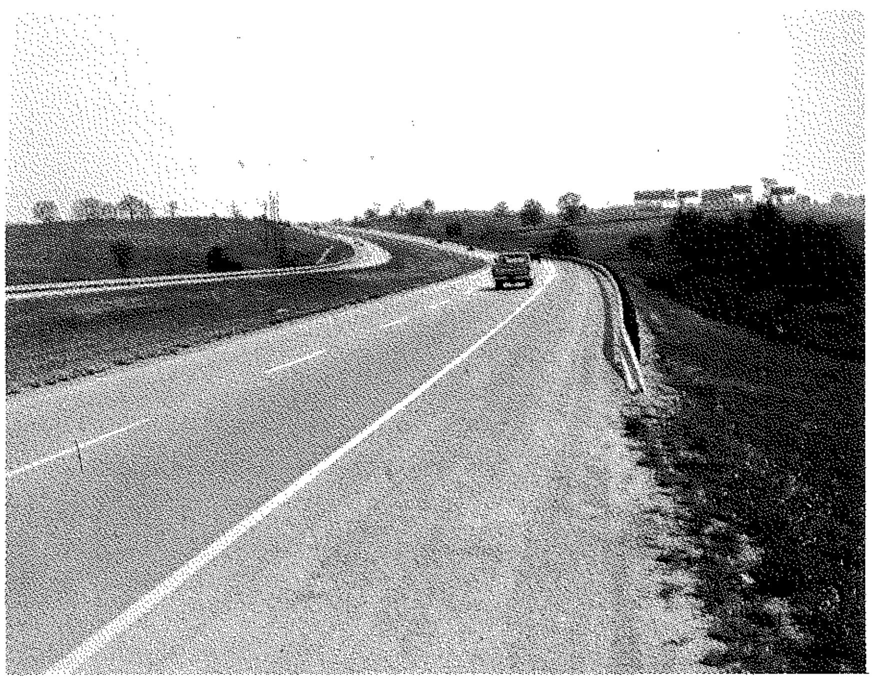


PAVEMENT DESIGN
0.1 GAL/S.Y. TACK - AS NEEDED

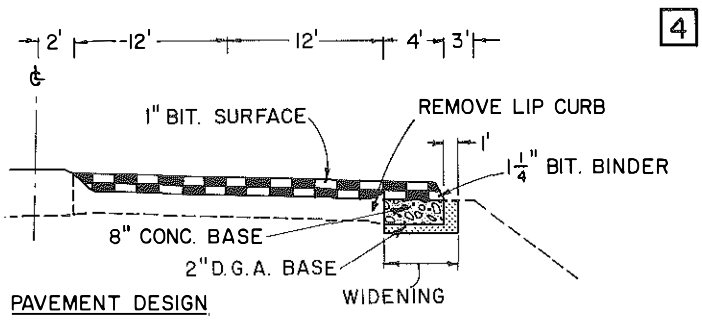
PAVEMENT
1000 TONS/MI. BIT. CONC. SURF.
FOR LEVELING & WEDGING
2" BIT. CONC. SURFACE (2-1" COURSES)

SHOULDERS
0.30 GAL./S.Y. SEAL COAT - AS NEEDED
20 LBS./S.Y. CR. AGGR. - AS NEEDED
SHOULDER - INSIDE
$1\frac{3}{4}$" BIT. CONC. SURFACE (1 OR 2 COURSES)
SHOULDER - OUTSIDE
$1\frac{1}{4}$" BIT. CONC. SURFACE AVE. ($1\frac{3}{4}$" INSIDE
$\frac{3}{4}$" OUTSIDE)

I-75-4 (32) 90
I-75-3 (36) 87 FAYETTE-MADISON CO. - KY. 876 TO NEAR GRIMES MILL RD.
13.1 MILES



Fayette-Madison Counties; I 75; From KY 876 to Grimes Mill Road; Resurfaced in 1971; Recent Photo.



½ TYPICAL SECTION

WIDENING
2" COMPT. DEPTH D.G.A. BASE
8" UNIF. DEPTH CEM. CONC. BASE

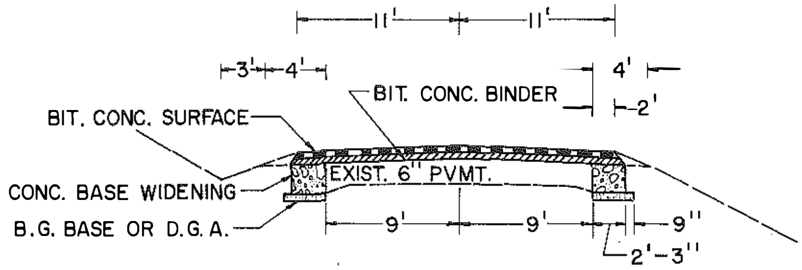
OVERALL SURFACING
0.1 GAL./S.Y. TACK
5 LBS./ SQ. YD. NAT. SAND (BLOTTER)
1¼" COMP. DEPTH BIT. CONC. BINDER
1" COMP. DEPTH BIT. CONC. SURFACE

SHOULDER (ONE FOOT WIDE)
12¼" COMP. DEPTH D.G.A.
5 LBS. /SQ.YD. CAL. CHD.

SP 150-95
SP 34 - 164 WOODFORD-FAYETTE COUNTIES-VERSAILLES-LEX. 7.202 MI.

Fayette-Woodford Counties; US 60; From Lexington to Versailles; Widened and Overlaid in 1965; Recent Photo.

5



PAVEMENT DESIGN (U.S. 51)

WIDENING

3" COMPT. DEPTH LOCAL BANK GRAVEL BASE
3" COMPT. DEPTH D.G.A. (BALLARD COUNTY)
9" P.C.C. BASE

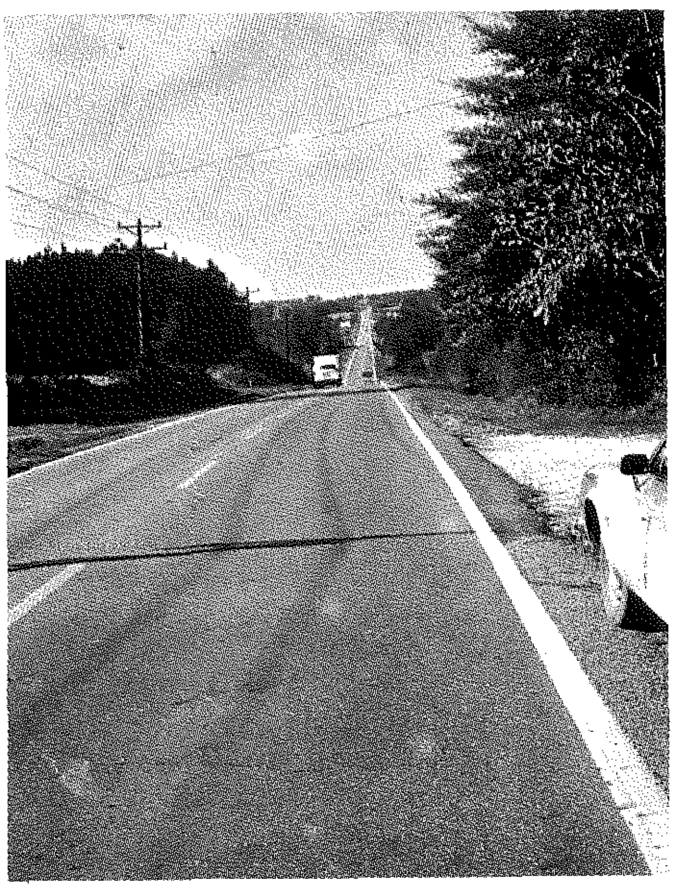
OVERALL

0.10 GAL./SY. TACK COAT
$1\frac{1}{2}$" AND $1\frac{3}{4}$" COMPT. DEPTH BIT. CONC. BINDER ($1\frac{3}{4}$" FULTON COUNTY)
$1\frac{1}{4}$" COMPT. DEPTH BIT. CONC. SURFACE

S.P. 4-61-4 BALLARD CO. FROM WICKLIFFE TO CARLISLE CO. LINE, 3.390 MILES.

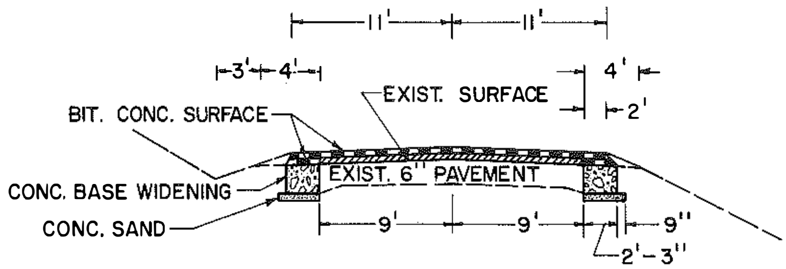
S.P. GROUP 12-1966, S.P. 20-4 CARLISLE CO., S.P. 53-9 HICKMAN CO. FROM BARDWELL TO CLINTON (EXCLUDING 1.8 MILES AT COUNTY LINE) 12.508 MILES.

S.P. GROUP 57-1962 FULTON CO. FROM HICKMAN CO. LINE (EXCLUDING THRU FULTON) TO TENNESSEE STATE LINE, 5.446 MILES.



Ballard-Carlisle-Hickman-Fulton Counties; US 51; From Wickliff to Fulton; Widened and Overlaid in 1962-1966; Recent Photo.

5A



PAVEMENT DESIGN (U.S. 51)

WIDENING

3" CONC. SAND

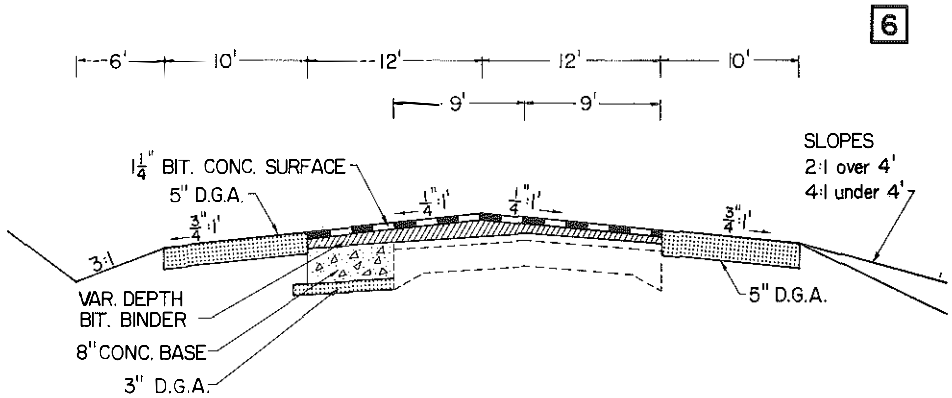
9" P.C.C. BASE

OVERALL

0.10 GAL./SQ. YD. TACK COAT

$1\frac{1}{2}$" COMPT. DEPTH BIT. CONC. SURFACE

SP 53-129-4 HICKMAN CO. FROM SOUTH CORPORATE LIMITS OF CLINTON TO FULTON CO. LINE. 6.358 MILES



PAVEMENT DESIGN

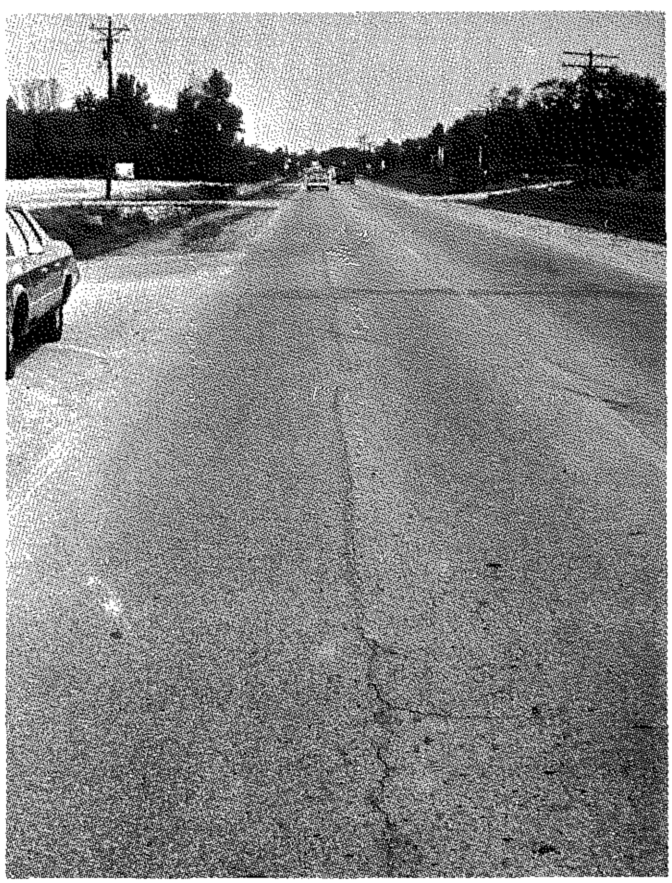
WIDENING
3" COMPT. DEPTH D.G.A.
8" UNIF. DEPTH P.C.C. BASE

OVERALL SURFACING
VAR. DEPTH BIT. CONC. BINDER
EQUIVALENT TO 2¼"
1¼" COMPT. DEPTH BIT. CONC. SURFACE
0.1 GAL./S.Y. TACK

SHOULDERS
5" COMPT. DEPTH D.G.A. BASE
2½ LBS./S.Y. CALCIUM CHLORIDE

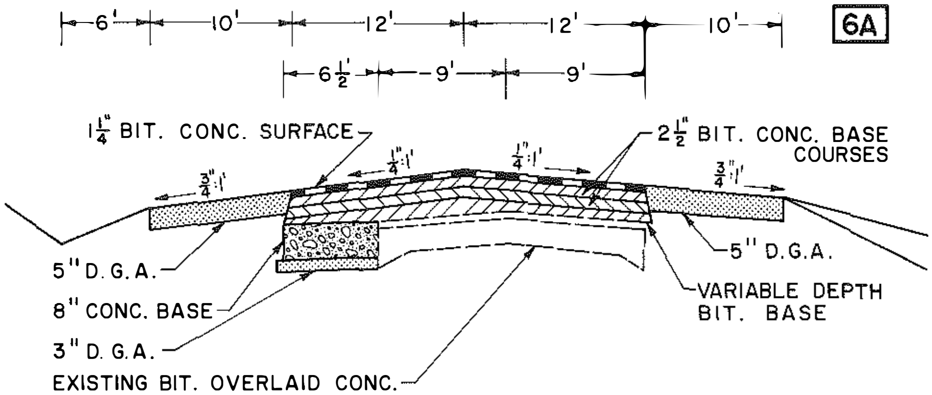
NOTE: ADD SUFFICIENT TONAGE FOR GRADER PATCHING OVER EXIST. PVT.

F 105(12) DAVIESS COUNTY-OWENSBORO-LEWISPORT- 3.93 MILES



Daviess County; US 60; From Owensboro to Maceo; Widened and Overlaid in 1965; Fonr-mile (6.4-m) Section; Recent Photo.

6A



PAVEMENT DESIGN

WIDENING

3" COMPT. DEPTH D.G.A. BASE
8" UNIF. DEPTH P.C.C. BASE

OVERALL SURFACING

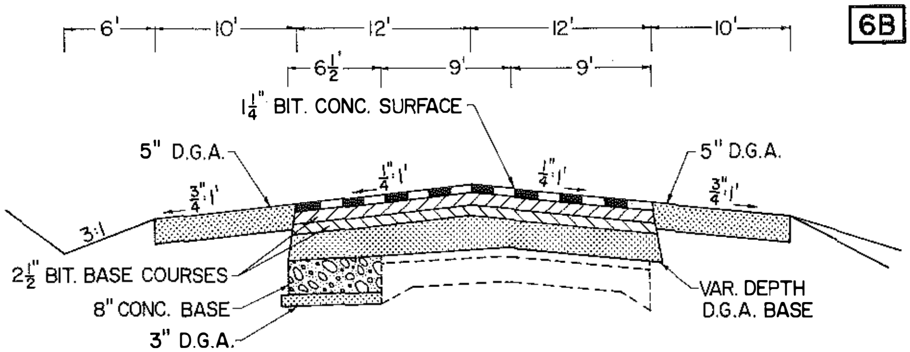
5" TO 8" BIT. CONC. BASE
$1\frac{1}{4}''$ BIT. CONC. SURFACE
0.1 GAL./S.Y. TACK

SHOULDERS

5" COMPT. DEPTH D.G.A. BASE
$2\frac{1}{2}$ LBS./S.Y. CALCIUM CHLORIDE

F 105(12) DAVIESS COUNTY-OWENSBORO-LEWISPORT – 0.12 MILES

6B



PAVEMENT DESIGN

WIDENING

3" COMPT. DEPTH D.G.A. BASE
8" UNIF. DEPTH P.C.C. BASE

OVERALL SURFACING

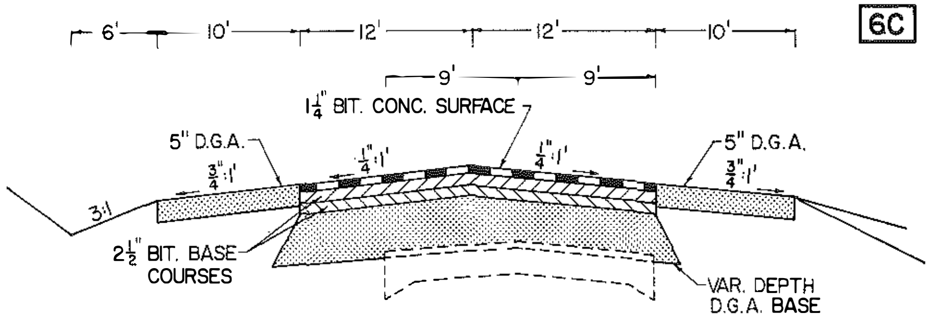
VAR. DEPTH D.G.A. BASE -3" TO 12"
5" COMPT. DEPTH BIT. CONC. BASE
$1\frac{1}{4}''$ COMPT. DEPTH BIT. CONC. SURFACE
0.1 GAL./S.Y. TACK

SHOULDERS

5" COMPT. DEPTH D.G.A. BASE
$2\frac{1}{2}$ LBS./S.Y. CALCIUM CHLORIDE

F 105(12) DAVIESS COUNTY-OWENSBORO-LEWISPORT - 0.197 MILES

6C



PAVEMENT DESIGN

PAVEMENT

VARIABLE DEPTH D.G.A. BASE - 12" TO 18"

5" COMPT. DEPTH BIT. CONC. BASE - (2-2½" COURSES)

1¼" COMPT. DEPTH BIT. CONC. SURFACE

0.1 GAL./S.Y. TACK

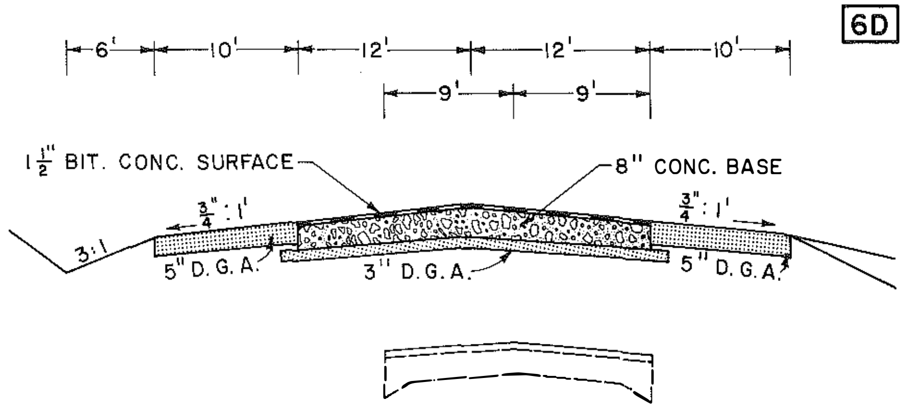
SHOULDERS

5" COMPT. DEPTH D.G.A. BASE

2½" LBS./S.Y. CALCIUM CHLORIDE

F 105(12) DAVIESS COUNTY-OWENSBORO-LEWISPORT- 0.161 MILES

6D



PAVEMENT DESIGN

PAVEMENT

3" COMPT. DEPTH D.G.A.

8" UNIF. DEPTH P.C.C. BASE

1½" COMPT. DEPTH BIT. CONC. SURFACE

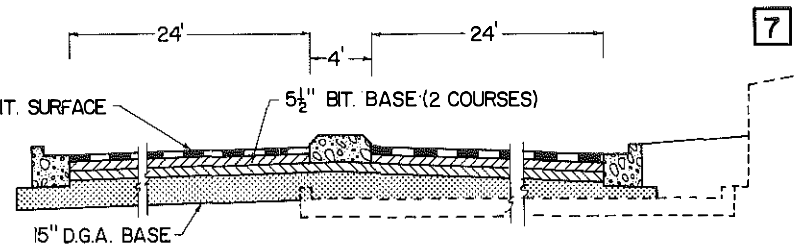
0.1 GAL/S.Y. TACK

SHOULDERS

5" COMPT. DEPTH D.G.A. BASE

2½ LBS./S.Y. CALCIUM CHLORIDE

F 105 (12) DAVIESS COUNTY - OWENSBORO - LEWISPORT - 1.24 MILES



PAVEMENT DESIGN

SALVAGE OVER OLD PAVEMENT

VARIABLE DEPTH D.G.A. BASE (24" MAX.)

WIDENED PORTION

15" COMPT. DEPTH D.G.A. BASE

OVERALL SURFACING

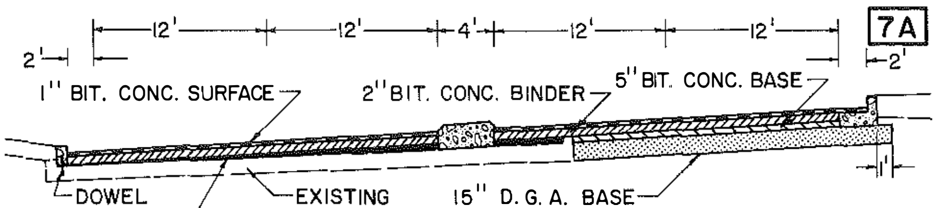
$5\frac{1}{2}$" COMPT. DEPTH BIT. BASE (2-$2\frac{3}{4}$" COURSES)

1" COMPT. DEPTH BIT. SURFACE

0.1 GAL./S.Y. TACK

NOTE: REMOVE EXISTING CURBS

APD 537 (51) BOYD CO. - CATLETTSBURG-ASHLAND RD. 1.49 MILES



PAVEMENT DESIGN

PAVEMENT

SALVAGE OVER EXISTING PAVEMENT

VARIABLE DEPTH BIT. CONC. BASE, BINDER AND/OR SURFACE FOR LEVELING AND WEDGING.

WIDENING

15" COMPT. D.G.A. BASE

5" COMPT. DEPTH BIT. CONC. BASE (2 - $2\frac{1}{2}$" COURSES)

OVERALL SURFACING

2" COMPT. BIT. CONC. BINDER

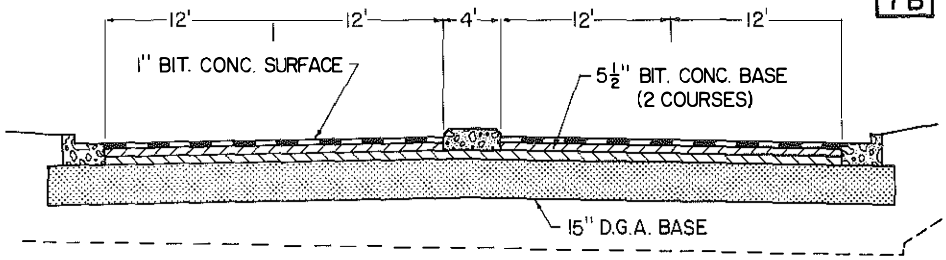
1" COMPT. BIT. CONC. SURFACE

0.10 GAL. / SQ. YD. TACK

NOTE: REMOVE EXIST. CURBS AND DOWEL HEADER CURB TO EXISTING PAVEMENT ON LEFT.

A'PD 537 (51) BOYD CO. CATLETTSBURG-ASHLAND ROAD 1.45 MILES

7B



PAVEMENT DESIGN

PAVEMENT

15" COMPT. DEPTH D.G.A. BASE

$5\frac{1}{2}$" COMPT. DEPTH BIT. BASE (2-$2\frac{3}{4}$" COURSES)

1" COMPT. DEPTH BIT. SURFACE

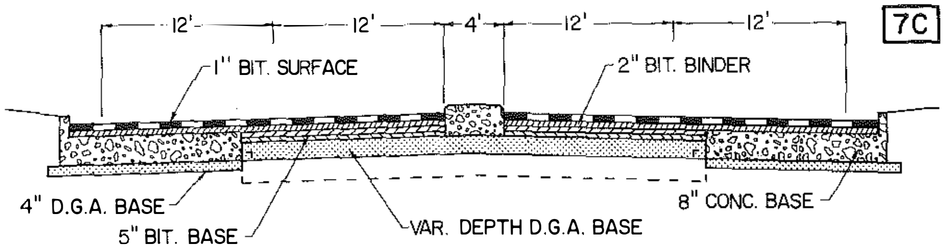
0.1 GAL./S.Y. TACK

NOTE: REMOVE EXISTING CURBS

BREAK-UP EXISTING PAVEMENT AND LEAVE IN PLACE

APD 537 (51) BOYD CO. - CATLETTSBURG-ASHLAND RD. .87 MILES

7C



PAVEMENT DESIGN

SALVAGE OVER EXISTING PAVEMENT

VARIABLE DEPTH D.G.A. BASE

5" COMPT. DEPTH BIT. BASE (2-$2\frac{1}{2}$" COURSES)

WIDENING

4" COMPT. DEPTH D.G.A. BASE

8" UNIFORM DEPTH CONCRETE BASE

OVERALL SURFACING

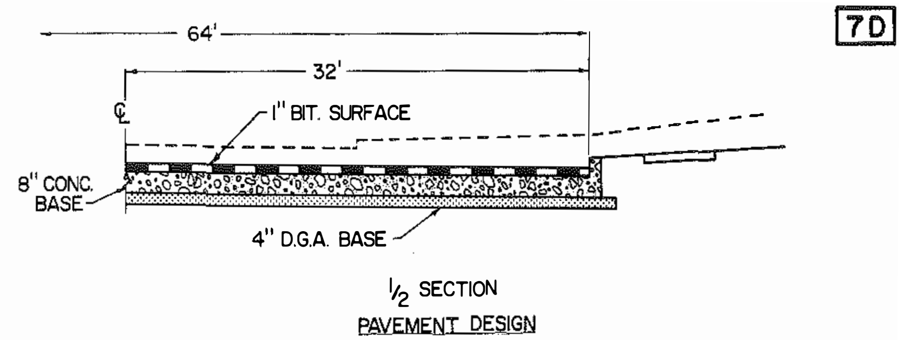
2" COMPT. DEPTH BIT. BINDER

1" COMPT. DEPTH BIT. SURFACE

0.1 GAL./S.Y. TACK

NOTE: REMOVE EXISTING CURBS

APD 537 (57) (58) BOYD CO.-CATLETTSBURG-ASHLAND RD. - 0.28 MILES



PAVEMENT

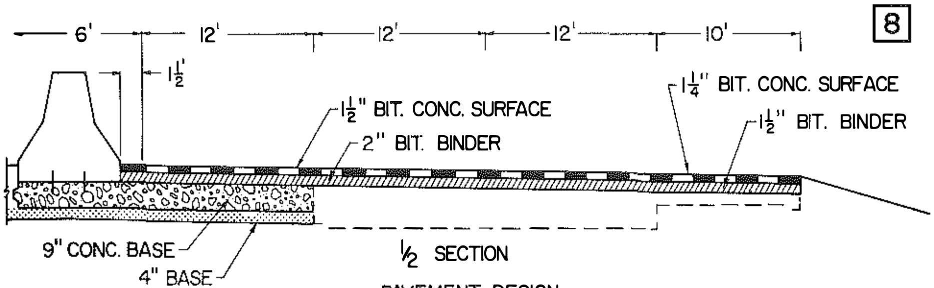
4" COMPT. DEPTH D.G.A. BASE

8" UNIFORM DEPTH CEMENT CONCRETE BASE

1" COMPT. DEPTH BIT. SURFACE

0.1 GAL./S.Y. TACK

APD 537 (57)(58) BOYD CO. - CATLETTSBURG-ASHLAND RD. 0.23 MILES



PAVEMENT DESIGN

STAGE ONE SURFACING

WIDENING

4" COMPT. DEPTH D.G.A.
9" UNIF. DEPTH P.C.C. BASE

OVERALL SURFACING

0.1 GAL./S.Y. TACK
BIT. CONC. BINDER AND/OR SURFACE FOR LEVELING
2" COMPT. DEPTH BIT. BINDER
1½" COMPT. DEPTH BIT. SURFACE

SHOULDERS (OUTSIDE)

0.1 GAL./S.Y. TACK
1½" AVG. COMPT. DEPTH BIT. BINDER
1¼" COMPT. DEPTH BIT. SURFACE
0.35 GAL./S.Y. SEAL COAT
20 LBS/S.Y. CRUSHED LIMESTONE

STAGE TWO SURFACING

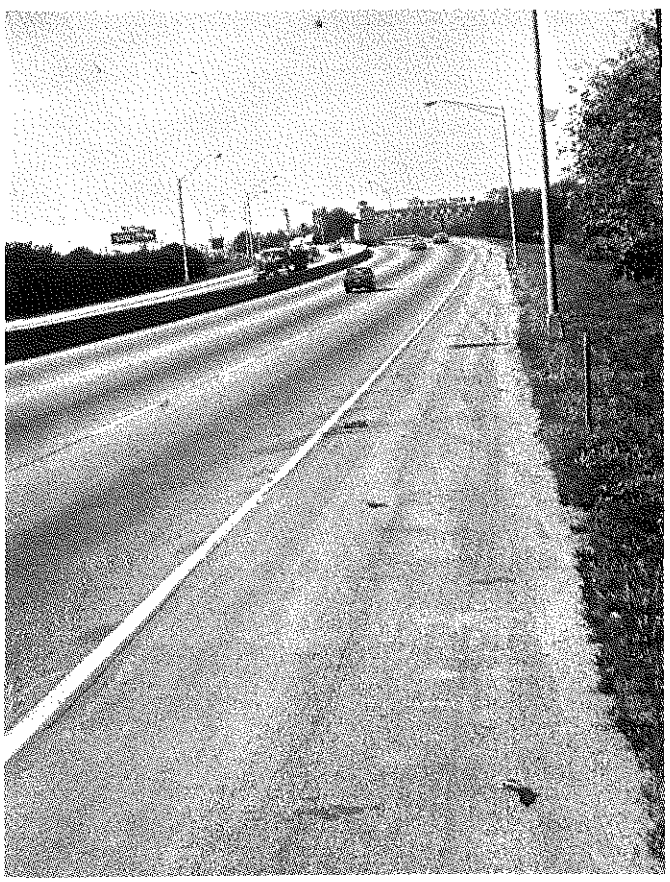
PAVEMENT

0.1 GAL./S.Y. TACK
1¼" COMPT. DEPTH BIT. SURFACE

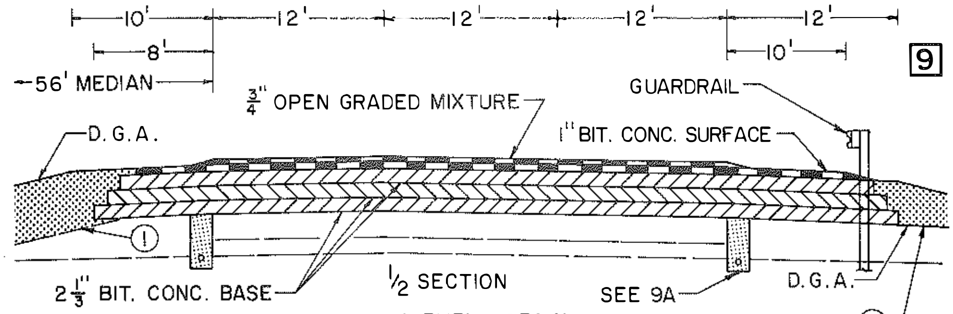
SHOULDERS

0.1 GAL./S.Y. TACK
1" COMPT. DEPTH BIT. SURFACE
0.35 GAL./S.Y. SEAL COAT
20 LBS./S.Y. CR. LIMESTONE

I-65-6(34)130 JEFFERSON CO. - NORTH-SOUTH EXPRESSWAY - 17,424 L.F. : 3.3 MILES



Jefferson County; I 65; North-South Expressway; From Watterson Expressway to Woodbine; Inside Lanes, Median Barrier, and Overlay Added in 1970; Recent Photo.



PAVEMENT DESIGN

0.10 GAL. TACK / SQ. YD. (AS NEEDED)

PAVEMENT

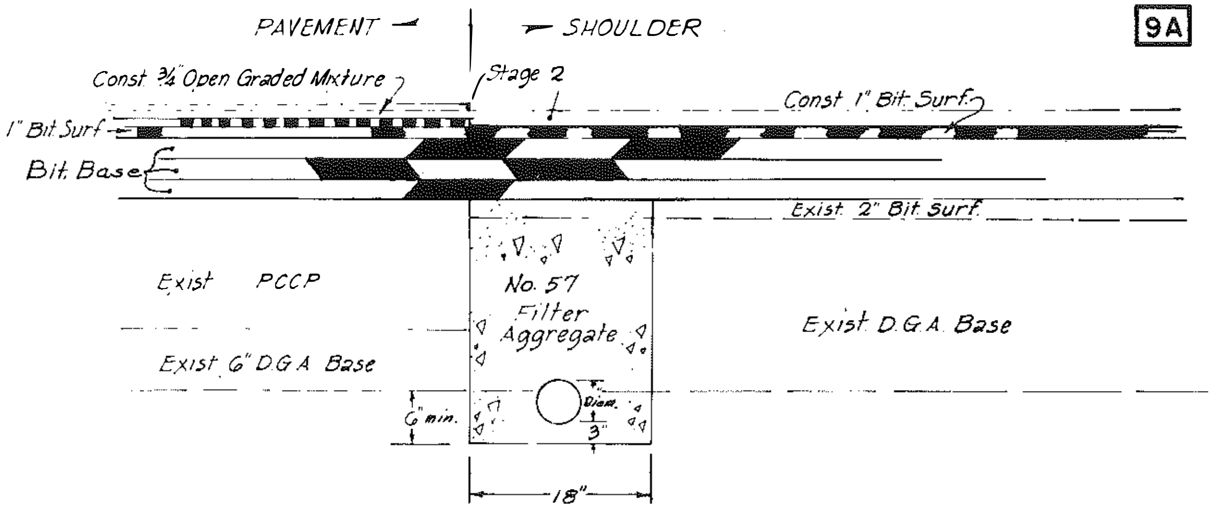
3/4" COMPT. BIT. OPEN GRADED MIXTURE
1" COMPT. BIT. CONC. SURFACE
7" COMPT. BIT. CONC. BASE (3 EQUAL COURSES)
1,000 TONS / MILE BIT. CONC. SURFACE FOR LEVELING & WEDGING

SHOULDERS

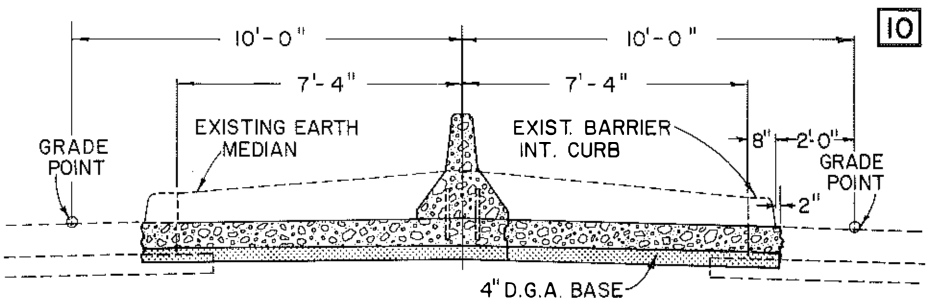
1" COMPT. BIT. CONC. SURFACE
7" COMPT. BIT. CONC. BASE (3 EQUAL COURSES)
(1) D. G. A. BASE IN THIS AREA

F I 75 – 7 (53)(54)(55)(56) BOONE CO. NORTH OF I-71 INT. TO U.S. 42 , 6.25 MILES

Boone County; I 75; From I 71 to US 42; Shows R-R-R Work in Progress; Lane-wide Patch Covers Area Fragmented after Proof-rolling; Narrow Patch Covers New, Plastic Drain Pipe; Inner Lane and Concrete Barrier Will Be Added.



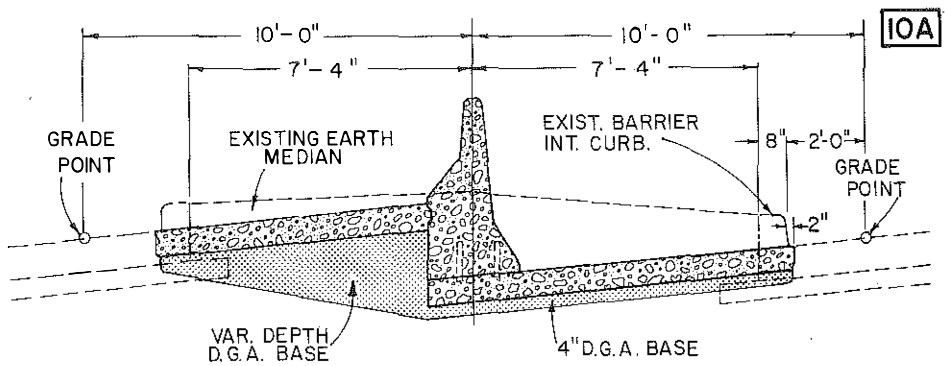
Note Longitudinal Pipe Drainage shall conform to Section 705 of the Standard Specifications except as noted on this detail



PAVEMENT

RIGID PAVEMENT AND MEDIAN BARRIER WALL
4" COMP. DENSE GRADED AGGR. BASE
8" UNIF. CEM. CONC. PVMT.

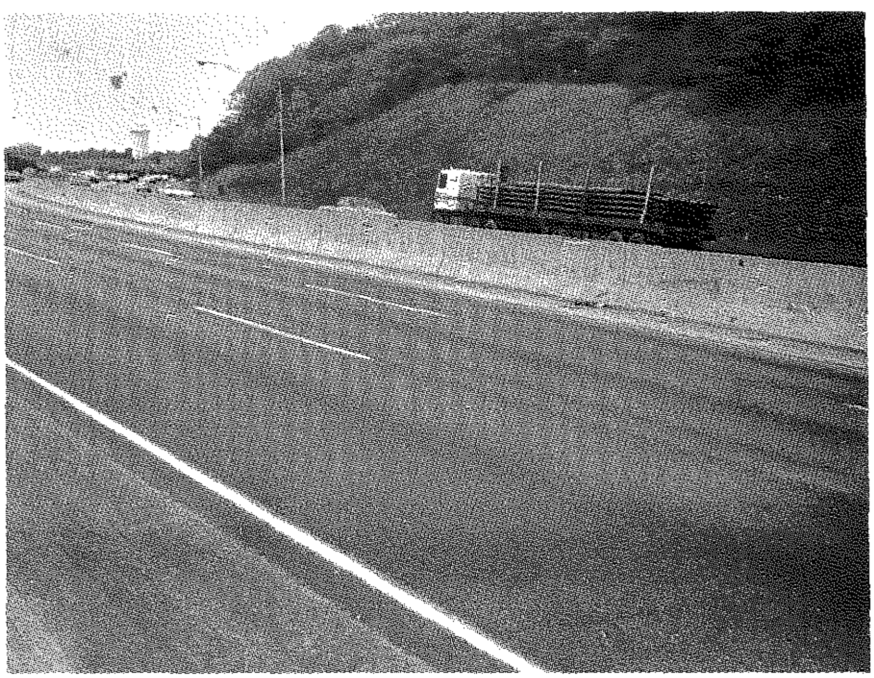
I 75-8 (25) 176 KENTON CO. COVINGTON - TENN. ST. LINE RD.- 3.78 MILES



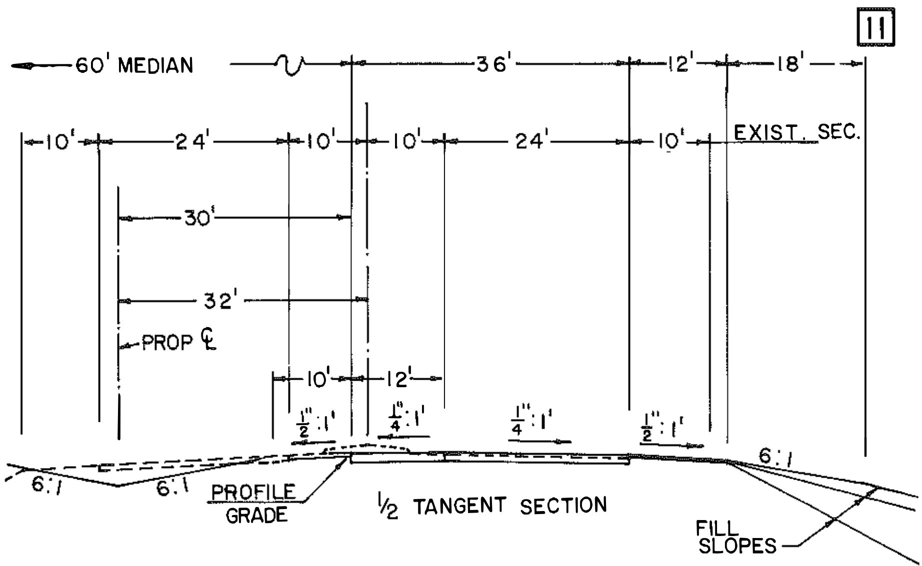
PAVEMENT

RIGID PAVEMENT AND MEDIAN BARRIER WALL
4" COMP. DENSE GRADED AGGR. BASE
8" UNIF. CEM. CONC. PVMT.

I 75-B (25) 176 KENTON CO. COVINGTON-TENN. ST. LINE RD.- 3.78 MILES



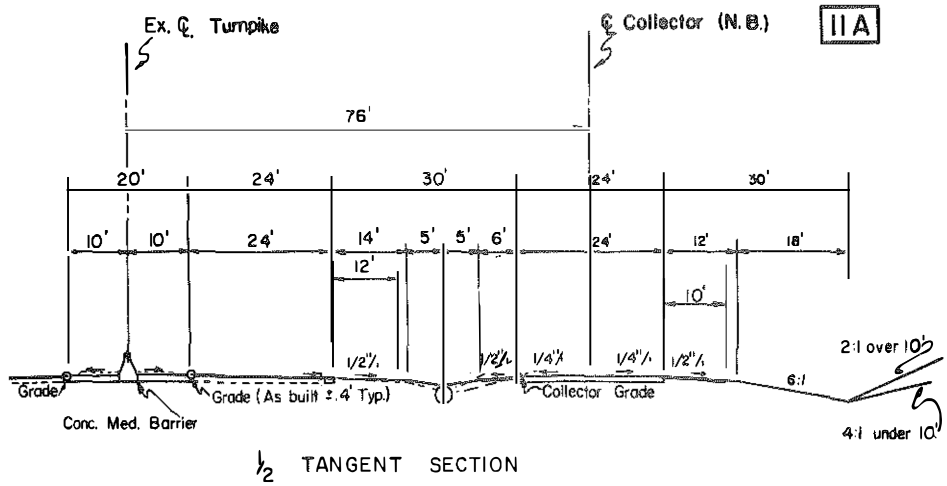
Kenton County; I 75 From US 25 - US 42 Interchange to the Ohio River; Revised in 1969; Recent Photo. Companion Photo Shows Open-graded Overlay Added to Northbound Lanes in 1976.



JEFFERSON COUNTY: KY. TURNPIK 165 FROM SOUTH OF W. KY. PKWY. INTCHG. AT E'TOWN TO NORTH OF THE BULLITT-JEFFERSON CO. LINE.

TP 15-574-43L
TP 47-69-29L
TP 56-688-30L

33.3 MILES



JEFFERSON CO. KY. TURNPIKE I 65, TP 56-688-30 L FROM SOUTH OF THE JEFFERSON FREEWAY INTCHG., EXTENDING NORTHWARD TO SOUTH OF WATTERSON EXPRESSWAY INTCHG.
4.09 MILES

Hardin-Bullitt-Jefferson Counties; I 65; From Elizabethtown to Louisville (Watterson Expressway); Retired Kentucky Turnpike; To Be Reconstructed; Recent Photo.