PERFORMANCE OF MICHIGAN'S POSTWAR CONCRETE PAVEMENT

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CONTENTS

SUMMARY

This study was conducted to determine the general level of performance of postwar pavements and, if possible, to statistically relate performance and certain design, construction, materials, and environmental factors. It was realized from the outset that many factors play a part in determining performance; that many of these factors are difficult to quantify, other factors difficult to measure or control. However, with the amount of data available (520 construction projects representing 1, 880 miles of pavement) it was felt that the more significant variables which affect performance might emerge from a multiple-regression type of statistical analysis. In addition, the study was made to determine specific points of weakness of design or construction procedures and to recommend possible means of improving future pavement performance.

Initial, 5-, 10-, and 15-year condition surveys, as well as roughness measurements taken with less regularity, were available to define performance of these postwar pavements.

It was found that information obtained by condition surveys, that is, transverse and longitudinal cracking, corner breaks, spalling, joint blowups, mud-jacking and patching, etc. were much more useful indicators of performance and remaining useful life of the pavement than roughness measurements. The general level of performance of these pavements after 10 and 15 years of service (1946-1954 construction), is given by the tabulation below indicating the medium performance level.

	Performance		
	10 years	15 years	
Transverse cracking, (99 foot slabs)	1.5	2.49	- cracks per slab
Longitudinal cracking	10	11.5	- feet per mile
Corner breaks, exterior corners	0.3	0.46	- percent of corners
Corner breaks, interior corners	0.2	0.2	- percent of corners
Spalling, exterior corners	12	29	- percent of corners
Spalling, interior corners	6	19	- percent of corners
Spalls along joint, not at corner	3.1	8	- per 100 joints
Joint Blowups, percent of projects with none	64	10	
Mud-jacking, percent of projects with none	78	----	
Patching, percent of projects with none	45	14	
Patching, median performance	less than 50	75	- sq ft per lane-mile

It should be stated that for most of these types of deterioration there was a tremendous spread in performance among the construction projects

studied. In general, most projects performed quite well but a small number of projects performed quite poorly. The most serious type of deterioration from the point of view of frequency was joint spalling, particularly corner joint spalling. One type of deterioration that was infrequent in the early life of the pavements, but became quite serious by the end of 15 years, was pavement joint blowups. It should be noted that there was a good correlation between percent of internal and external corner spalls at 5 years and blowups per 100 joints at the end of 15 years. Thus, early joint spalling is a good indicator of later more serious joint repair problems.

Three pavement performance models, Present Serviceability Index (PSI, from the AASHO Road Test), Performance Rating Factors (RPF) and Structural Deterioration Index (SDI) were used to evaluate the effect of seven possible causal factors in a multiple regression statistical analysis. Two of these seven factors were found to have a significant effect on pavement performance. These two factors were the percent of soft, non-durable content in the coarse aggregate and the average daily commercial traffic.

Since most projects performed satisfactorily, and the same basic design was used for all projects, it is apparent that causal factors for the poor performance of a few projects are much more likely to be related to materials, construction factors, or environmental factors of climate and traffic loading. All condition survey indicators of performance (transverse cracking, longitudinal cracking, external or internal corner spalls, deterioration, and patching) showed that the traffic lane (the lane with the most and heaviest traffic) had 65 percent poorer performance than the passing lane as measured by the Depreciation Index.

The report shows that performance indices, of either the subjective or objective type, based on condition surveys are much more valuable in indicating structural deterioration than is the Present Serviceability Index which is based primarily on roughness. Moreover, performance indices based on condition surveys serve to measure the "remaining useful life" of pavement while the Present Serviceability Index is nearly useless in this respect. Signs of short service life appear in the five-year condition surveys. These early signs are significantly correlated with later structural performance as measured by the ten and fifteen year surveys. Thus, after five years of service, it should be possible to determine which projects will fail prematurely.

Pavement joint blowup frequency is considerably higher for aggregates containing greater amounts of soft, non-durable material than it is for aggregates with lower amounts, thus blowups can be causally related to this type of deleterious content in the coarse aggregate.

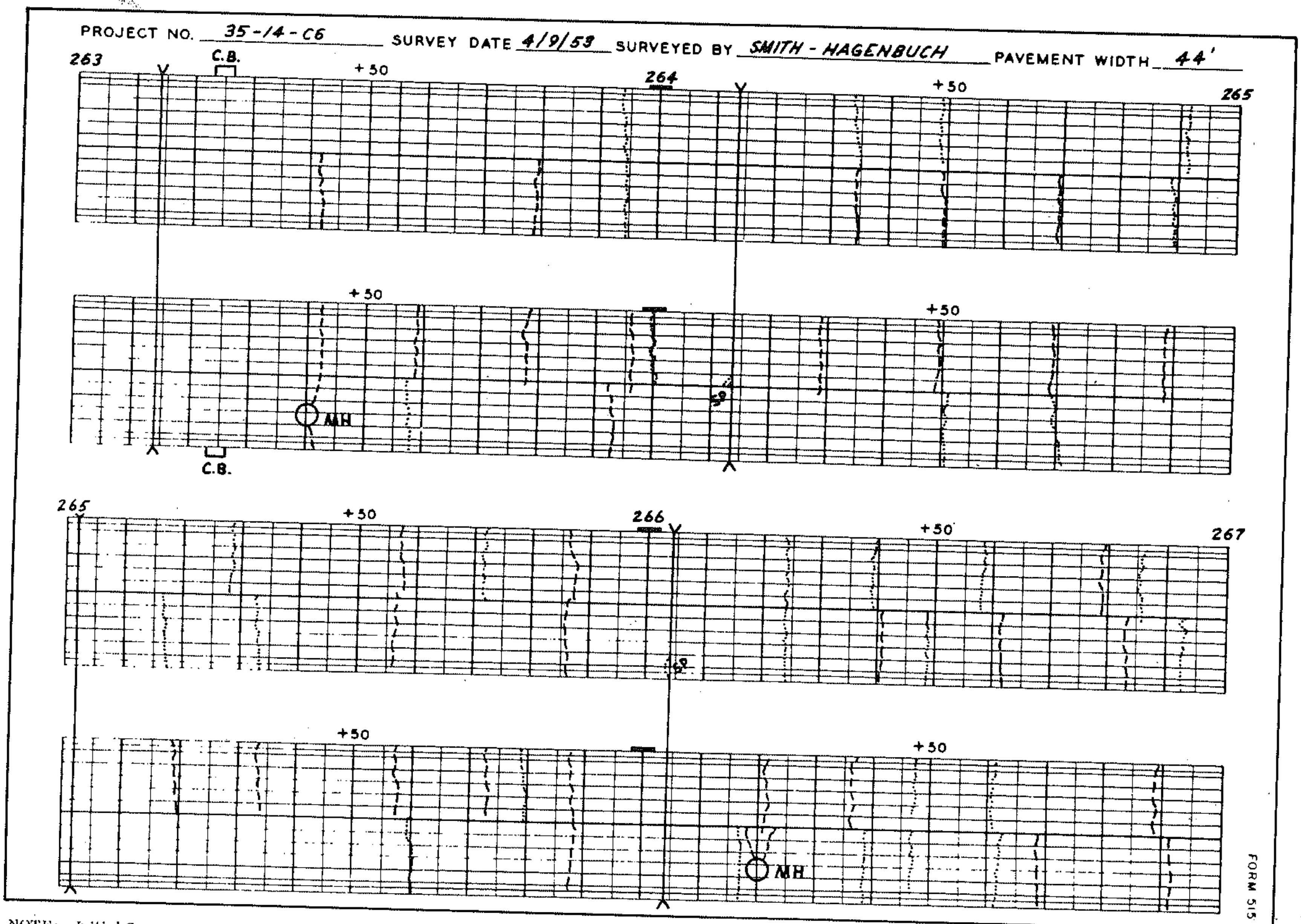
INTRODUCTION

This study was started in July 1963 in cooperation with the Bureau of Public Roads under the Highway Planning and Research Program. However, collection of the condition survey data which made evaluation of performance of all Michigan's postwar concrete pavements possible began in 1946, and initial roughness measurements of concrete pavements in 1949 after constructing a roughometer patterned after the Bureau of Public Roads model.

The primary purpose of this study is to obtain comprehensive information on the performance of pavements constructed since 1946, the beginning of Michigan's "postwar design" construction; a design that remains fundamentally the same today. It was anticipated that this detailed performance study would suggest certain changes in design or materials that could effect improvement in future pavement construction.

These postwar pavements were subjected to surveys shortly after construction and at five-, ten-, and fifteen-year periods in as systematic a pattern as scheduling limitations would allow. As mentioned in the "Scope" of the Proposal, the records and compilations of pertinent construction and materials information, along with condition and roughness surveys for a total of approximately 520 construction projects (1, 880 miles) contain the essential ingredients for a comprehensive statistical analysis on pavement performance and the factors that may influence this performance.

The proposal outlined two phases; a cause and effect study of pavement performance, followed by a second phase which would include analytical and experimental studies of the effect of certain design innovations on pavement performance. These innovations were to consist of various design improvements with a determination of whether improved performance as measured by durability, riding quality, and reduction in maintenance warranted the expected increase in initial construction costs. This second phase was also to include certain laboratory model studies to resolve design issues for conventional concrete pavements, and to determine if there were inherent performance advantages to prestressed concrete or continuously reinforced pavements for high-volume heavy axle-load situations.



NOTE: Initial Survey - There were no cracks or other defects at this time.
5-Year Survey - Cracks and other defects shown by -------------
10-Year Survey - Cracks and other defects shown by

Figure 1. Typical condition survey sheets. Color

The first phase was to run for twelve to eighteen months, and the second phase for fifteen months. The second phase was never started, since it was not possible to attain the research effort in two years of the program which had been scheduled for the first year. This was due to the necessity of assigning personnel to projects that presented more immediate demands.

However, with this curtailed program most of the objectives of Phase I have been met and will be discussed within the report. As stated in the Proposal, the specific objectives of Phase 1 were:

I - To determine the general performance of all postwar pavements.

II - To determine specific points of weakness of design methods or construction procedures.

III - To determine the effect of various sources of materials used in the projects on the performance characteristics of the pavement.

IV - To determine the relationship between pavement performance and quality of the subgrade support.

V - To investigate in detail the cause of certain specific types of pavement distress.

Condition surveys are made on each newly completed concrete pavement project as soon after completion as possible, within the limitations of scheduling. The survey data were collected by walking along the edge of the pavement in the direction of increased stationing, which is embossed in the pavement at 100-ft intervals. A measuring wheel recorded the distance of items to be noted between the 100-ft stationing points. The markings placed on the survey sheets in the field are made to scale for cracking, corner breaks, spalling, etc. (Fig. 1). Certain symbols are used to denote the type of condition. Definitions of terms for pavement condition surveys are given in Appendix I. Appendix I also illustrates photographically the typical types of deterioration that were observed. Periodic resurveys are generally made for most projects after five, ten, and fifteen years of service. The same survey sheets were used for subsequent surveys but the condition changes at these periods were noted by using a different colored pencil marking for each survey, making it possible to tell on which survey the first indication of deterioration was observed. Extensions of cracks are noted by elongation of the crack in the appropriate color.

In studying and comparing the condition of pavements in service it was decided to compare performance at the end of five, ten, and fifteen years. At the time the condition survey data were tabulated, it was possible to use the projects constructed during 1946 through 1948 for fifteen-year performance and the 1946 through 1958 projects for the five-year performance. However, preliminary study and analysis indicated that more significant results appeared possible after ten and fifteen years of service. Thus, it was decided to study only the five-year performance on those projects (1946 through 1953) which also had ten- or fifteen-year service records in order to determine the rate of pavement deterioration that had occurred. The condition survey data remain available but in untabulated form for the five-year surveys of projects constructed between 1953 and 1958. Initially, the general analysis was confined to 50 projects with fifteen-year service records representing 193.1 project miles and 207.9 miles of equivalent two-lane pavement; and 148 projects representing 510.1 project miles and 563.4 miles of equivalent two-lane pavement with both five- and ten-year service records. However, as the project progressed, additional surveys became available and, where possible, were incorporated in those aspects of the analysis not already completed.

The condition survey data were tabulated such that the performance of all projects could be evaluated on a common and equitable basis. This entailed a good deal of study for some performance variables. For example, the number of interior and exterior corner breaks cannot equitably be compared per mile of pavement for two-lane or four-lane undivided roadway. Also, such variables as the number of spalls at transverse cracks cannot be compared without also taking into account the variation in the number of cracks from one project to another.

Another source of basic data for this study was Research Laboratory Report No. R-235, "Compilation of Design and Construction Data for Concrete Pavements on the State Trunkline System," printed in October 1955. This book--in coded form--contains the essential design and construction data pertaining to concrete pavements constructed from 1919 through 1953. Some additional data pertinent to pavement performance were required for this study which were not as readily available, e.g., the quality of subgrade soil. These additional data had to be obtained by searching Department records on a project-by-project basis.

Another source of data was the annual roughometer survey of newly constructed concrete pavements. These roughness measurements have been made, since 1951, with a Research Laboratory roughometer patterned after the Bureau of Public Roads design. In addition to the initial roughness survey, measurements were taken of pavements in service. However, a regular pattern of such roughness measurements to tie-in with the condition surveys was not made until the last few years, when the matter of associating pavement roughness with performance as determined by condition surveys was planned in this study. All available prior roughness data were used, however, and where possible, roughness measurements were made on projects after ten or fifteen years of service in this latter period. All projects with fifteen years of service had roughness measurements taken between fourteen and sixteen years of service. However, initial roughness measurements for these projects were not available. For projects with ten years of service, those constructed since 1951 had initial roughness values and 57 of the total of 148 projects had roughness measurements between nine and eleven years of service.

Preliminary Analysis Studies

As mentioned previously it was not possible to obtain the pavement roughness history for the projects under study as completely as would have been desired. However, certain roughness data were available in terms of measurements taken at odd years rather than after five, ten, or fifteen years of service. Therefore, an analysis was conducted to determine if it was possible to interpolate or extrapolate roughness values from measured intervals, to intervals that would match those where condition survey records were obtained. If, with reasonable accuracy, the rate of roughness increase could be correlated with years of service, then such projections might increase the amount of valid data that could be analyzed. The relationship turned out to be completely unsatisfactory for such projection purposes since the variation in roughness increase was large. Another attempt was made, on the supposition that perhaps the absolute increase in roughness was related to some extent to the initial roughness value. The relationship between the absolute increase in roughness for five- or ten-year periods was compared with the initial roughness value. The maximum increase is 45 inches per mile for five years and 85 inches per mile for ten-year periods with average increases of 25 to 44 inches per mile, respectively. The average increase in roughness per year for the five-year period is 5.0 inches per mile and 4.4 inches per mile for the ten-year period. However, again the variation in the rate of increase of roughness is so great that interpolation or extrapolation of roughness data to other periods is hazardous. The best correlation, although not sufficiently good for projection, was obtained with respect to the relationship of increase in roughness as a percent of the initial roughness values. As a result of this preliminary analysis it was decided that if a roughness measurement was available for a project which was taken within one year of the time of the condition survey, it would be used in the analysis. The roughness measurement data for pavement projects taken between nine and eleven years of service were grouped with ten-year condition survey data and fourteen to sixteen years of service with fifteen-year condition survey records.

OBJECTIVES I AND II

GENERAL PERFORMANCE OF ALL POSTWAR PAVEMENTS AND SPECIFIC POINTS OF WEAKNESS OF DESIGN METHODS AND CONSTRUCTION PROCEDURES

One of the primary objectives of this study, and a foremost consideration at the time this pavement condition survey program was initiated in 1946, was to obtain a comprehensive measure of pavement performance. As a result of the Report, "The Design of Concrete Pavements for Postwar Construction," Research Laboratory Report No. R-68A, and the preliminary evaluations of experimental test roads built in Michigan, California, Kentucky, Minnesota, Missouri, and Oregon, a new design for concrete pavements in Michigan was adopted in 1946. Using this design concept, the pavement consisted of long reinforced slabs and contraction joints only. The need for expansion joints to prevent undue compression and pavement Blowups was no longer apparent as a result of the performance of these test roads where, in the case of Michigan's, the expansion joint spacings varied from 90 to 2,700 feet. Prior to this, various joint combinations had been used in Michigan, viz: 1) The exclusive use of 1-in. expansion joints spaced at 100 ft, 2) The use of contraction joints at close intervals, and 10, 20, and 30 ft and expansion joints at varying intervals, 3) The use of expansion, contraction and hinged or warping joints in a 120-60-30 combination (expansion joints spaced at 120 foot intervals, contraction joints midway between the expansion joints, and hinged joints midway between the expansion and contraction joints), thus resulting in 30-ft slabs.

Since 1946 the primary design features have been unchanged or changed only slightly. These features are as follows:

Pavement thickness - 9 in. uniform thickness.

Contraction joint spacing - 99 ft. In 1963 this was changed to 71 ft 2 in. but all projects under this study were constructed with a 99-ft slab length.

Expansion joint spacing - Used only at special locations, or when placing concrete before April 15, or after September 15, and then at 396-ft intervals.

Subbase - 12 in. of granular material.

Load Transfer - 1 in. diameter by 15-in. dowels at 12-in. centers.

Reinforcement - Mesh, bar mat, or expanded metal at approximately 76 to 88 lb per 100 sq ft, depending on lane width and type of reinforcement.

In 1953, the load transfer dowels were changed from 1 in. diameter to 1-1/4 in. diameter and the assemblies for supporting the dowels were upgraded to permit only those that held the dowels rigidly in position in order to eliminate dowel misalignment problems.

Since 1955 a selected subbase has been used directly beneath the concrete pavement to provide a smooth, durable, and firm surface for concrete and to provide better support for the load transfer assemblies. This has consisted of 3 in. of selected material in addition to 11 in. of regular subbase material resulting in a total subbase thickness of 14 in. Since 1963 the selected subbase has been 4 in. with 11 in. of regular subbase for a total thickness of 15 in.

PERFORMANCE OF POSTWAR PAVEMENTS AFTER 10 AND 15 YEARS OF SERVICE

While condition surveys were available for five-, ten-, and fifteen-year service periods, preliminary analysis indicated that performance differences were less distinct or significant at five years and, therefore, the reported performance will be confined to service periods of ten and fifteen years only.

Ten Year Service

The performance of pavements after ten years of service will be discussed in terms of the physical manifestations of deterioration that could be noted by periodic condition surveys.

1. Transverse Cracking

The number and percent of pavement slabs which had 0, 1, 2, 3, ... , n, transverse cracks per slab was tabulated for each project. Frequency distributions of the percent of slabs with 0, 1, 2, 3, ... , n transverse cracks per slab were graphed to determine the variations encountered for 76 projects. The range in performance can be clearly noted in Figure 2 where the average of the ten best and ten worst performing projects with respect to transverse cracking are plotted. The extreme difference in performance is very apparent. The transverse cracking for these ten best and ten poorest performing projects indicates that the extreme differences between good and bad performance is not due to isolated "flukes" but that the normal performance distribution simply covers a tremendous spread. The distribution of the average number of cracks per slab of all two-lane projects after ten years of service is illustrated in Figure 3. For these 82 projects the average varied from 0.03 to an average of 5.17 cracks per slab. The median performance was 1.50 cracks per slab, and approximately 80 percent of the projects had less than 2.8 transverse cracks on the average per slab. For the same 82 projects with ten years of service the frequency distribution, by project, of the percent of slabs of that project with no transverse cracks is also shown in Figure 3. The median performance project had 38.6 percent of its slabs uncracked while 13 percent of the projects had more than 70 percent of their slabs uncracked after ten years of service.

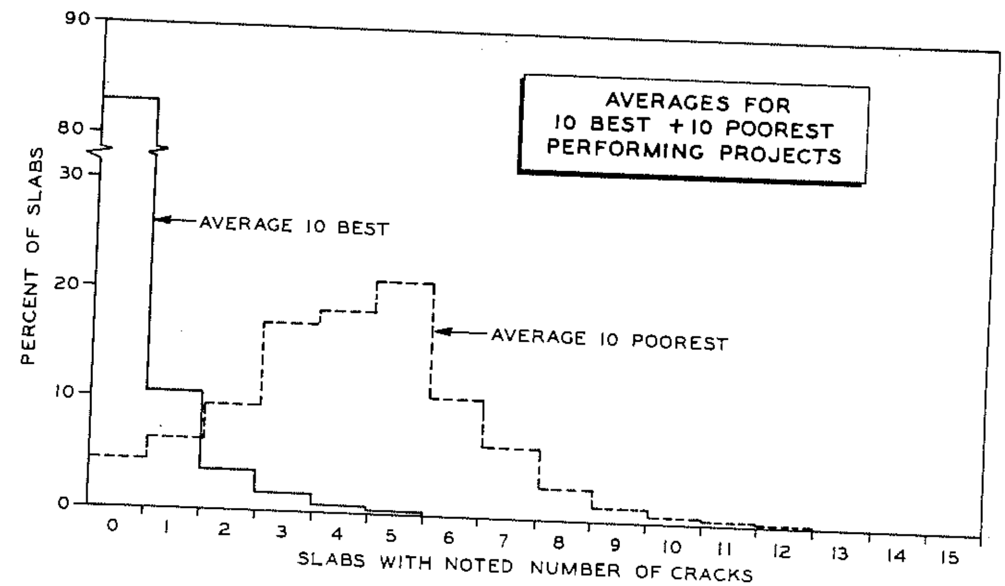


Figure 2. Histogram of transverse cracks per slab for two-lane pavement; 10 years of service.

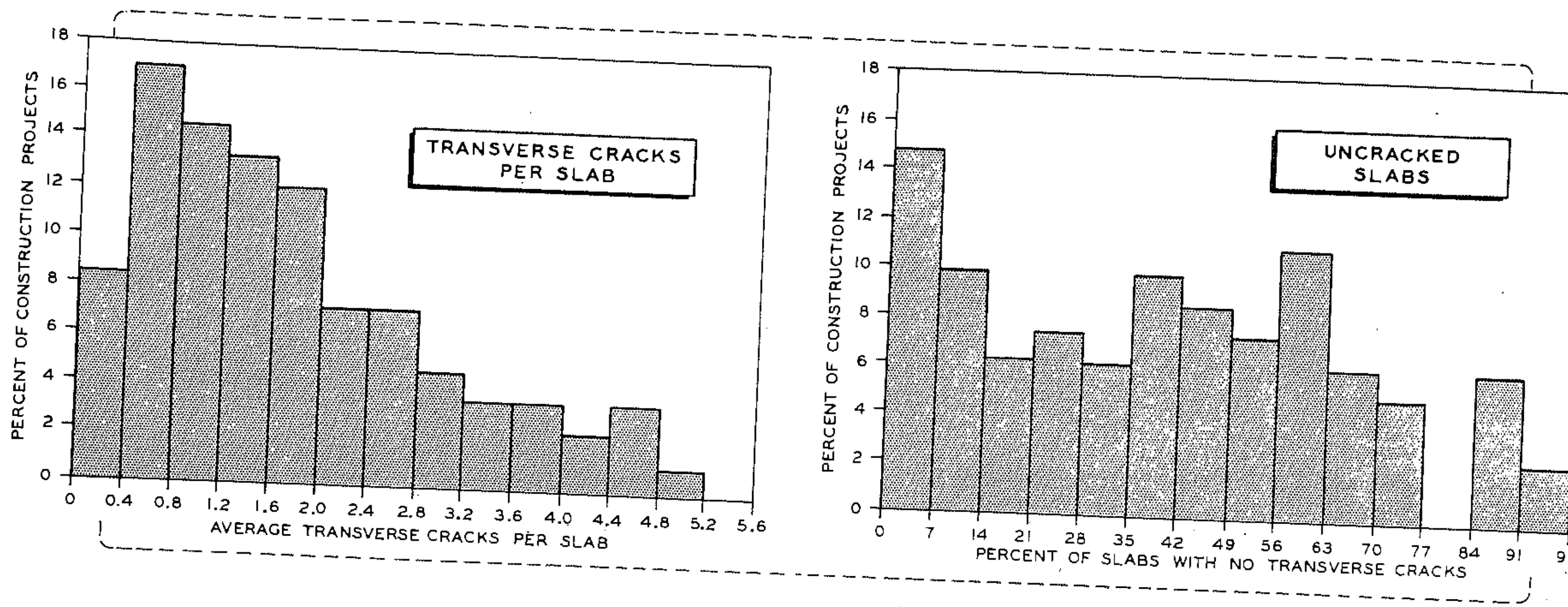


Figure 3. Histograms of transverse cracking for 82 two-lane pavement projects; 10 years of service.

2. Longitudinal Cracking

The distribution of longitudinal cracking among 124 construction projects with ten years of service for two-lane, or four-lane divided pavements, is shown in Figure 4. Construction specifications called for 1/4- by 2-in. bituminous filler strip to form the plane-of-weakness for the centerline crack on standard construction projects from 1946 through 1954. Since 1954, the centerline construction joint has been sawed a minimum of 1/8 in. wide to a minimum depth of 2 in. For all projects studied with ten years of service the bituminous filler strip was used to form the centerline. A wide difference in performance can be noted in Figure 4; for 49 percent of the projects the longitudinal cracking is between 0 and 9 ft per mile of equivalent two-lane pavement, but for seven projects, or approximately 5.5 percent of the total, the cracking was 100 ft or more per mile of pavement. The worst performing project had cracked at the rate of 266 ft per mile, or approximately 5 percent of its length.

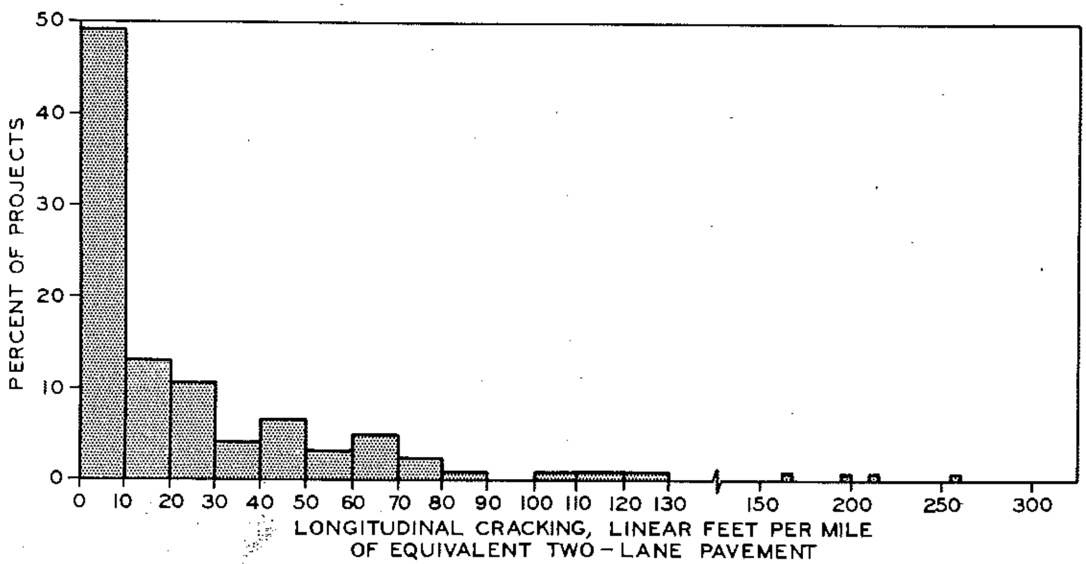


Figure 4. Histogram of longitudinal cracking for two-lane and four-lane divided highway construction projects with 10 years of service (124 projects).

During 1953, on one construction project, short stretches of pavement (between 1,000 and 2,000 ft) were sawed to depths of 1, 1-1/4, 1-1/2, or 1-3/4 in., instead of the standard depth (a minimum of 2 in.) to determine the effect of this on longitudinal cracking. This experiment indicates that when the depth of cut is reduced to 1-3/4 in., the longitudinal cracking as a percentage of pavement length increased approximately four times, and that for a 1-in. cut the longitudinal cracking is approximately 25 times as great as for the standard 2-in. minimum cut (Fig. 5).

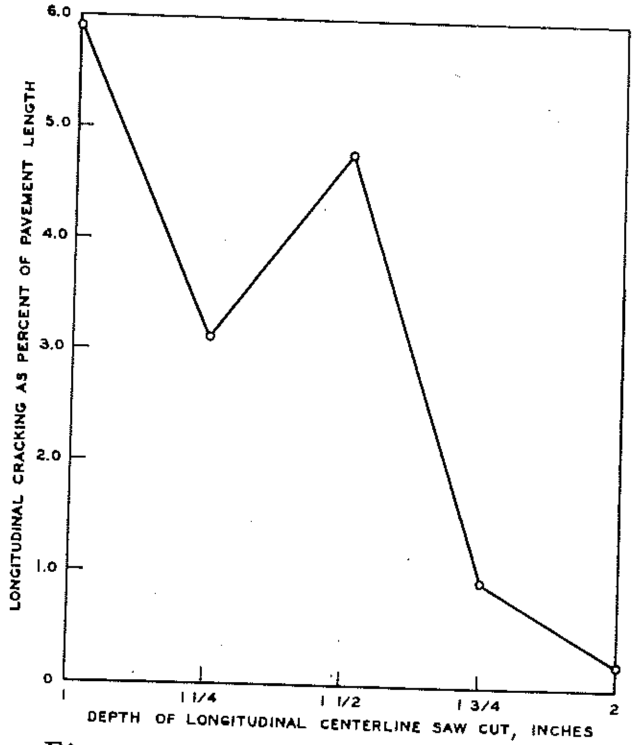


Figure 5. Effect of longitudinal centerline saw-cut depth on subsequent formation of longitudinal cracking; 10 years of service.

3. Corner Breaks

Figure 6 is a histogram showing the distribution of exterior and interior corner breaks for 82 two-lane construction projects with ten years of service. Twenty-nine percent of the projects had less than 0.1 percent of the possible exterior corners broken and forty-six percent of the projects had less than 0.1 percent of the possible interior corners broken. This good performance, however, is offset by 17 projects, or slightly more than 20 percent, where more than 1 percent of the exterior corners were broken, and eight projects (approximately 10 percent) with more than 1 percent of the interior corners broken. The extreme of poor performance was 44.8 percent of the exterior corners broken for one project and 13.9 percent of the interior corners for another. This is a serious problem for a few individual projects but the vast majority of the projects are performing quite well.

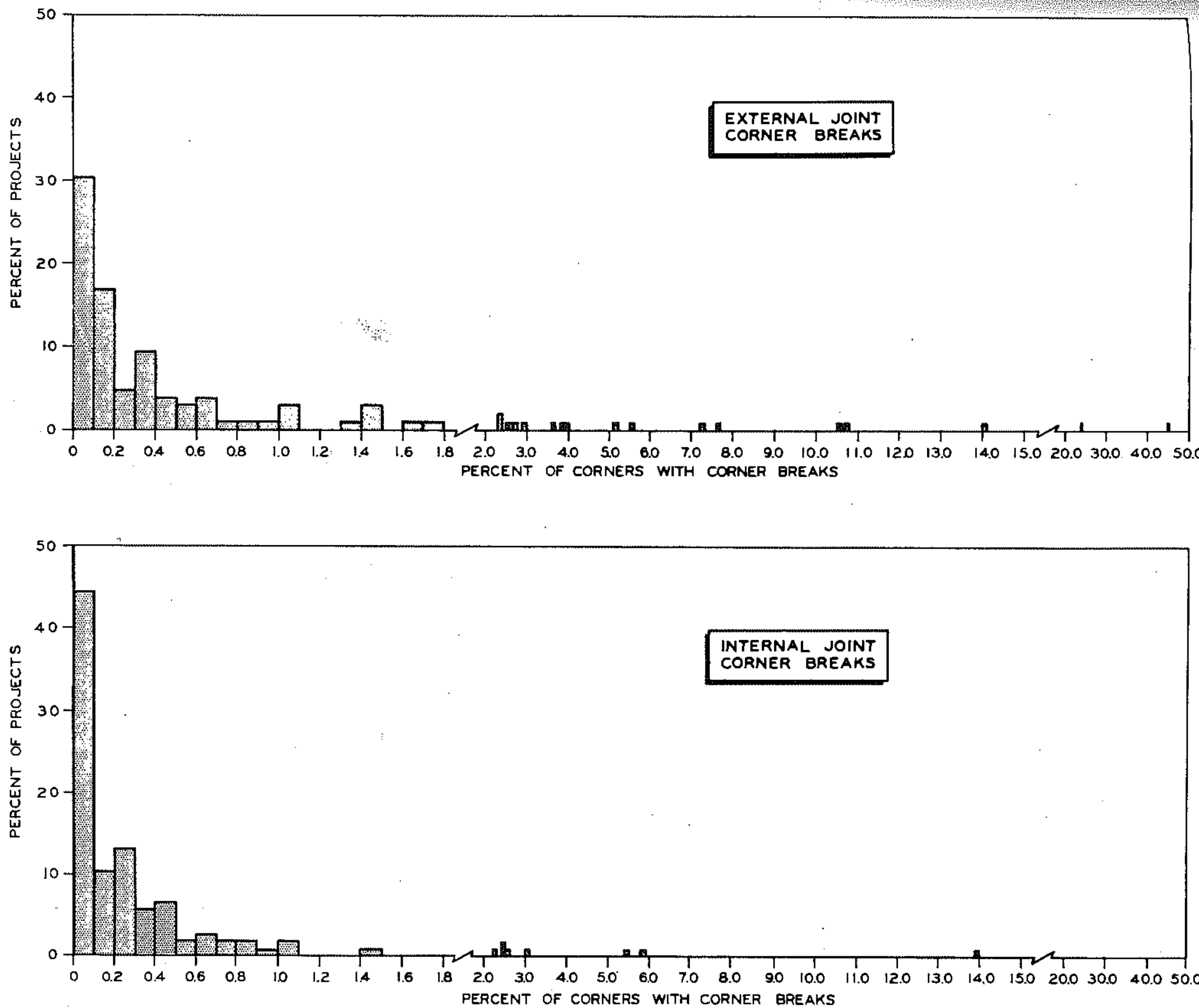


Figure 6. Histogram of corner breaks for 82 two-lane pavement projects; 10 years of service.

The distribution of corner breaks at both exterior and interior corners at transverse cracks is shown in Figure 7. Thirty-four percent of the projects had no corner breaks at transverse cracks and 86 percent had less than one corner break at a transverse crack for 100 cracks. The poorest performing project had 3.8 corner breaks at transverse cracks per 100 cracks.

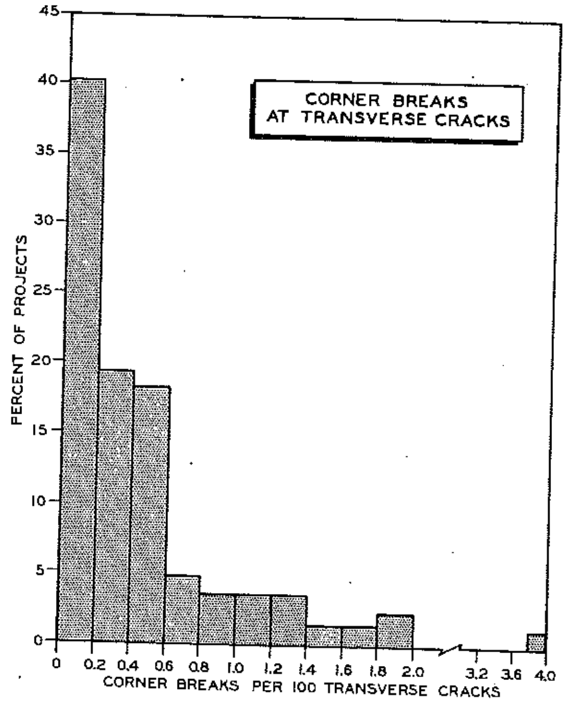


Figure 7. Histogram of corner breaks at transverse cracks for 82 two-lane pavement projects; 10 years of service.

4. Spalls

Spalling has been characterized in the following ways: 1) spalls at external corners, 2) spalls at internal corners, 3) spalls along transverse joints (not at joint corners), 4) spalls along transverse cracks, 5) spalls along longitudinal centerline joint, 6) spalls along outside longitudinal edge of slab, and 7) spalls in interior surface of slab. The distribution by construction projects of spalls at external and internal joint corners is shown in Figure 8. Pavement performance has not been particularly good for this indicator of pavement deterioration since 36 projects (44 percent) had more than 15 percent of the possible external corners spalled after ten years of service and the poorest performing project had 43.3 percent spalled. As might be expected, spalling at interior corners is generally not as severe as at external corners. Slightly over 30 percent of the projects had less than 3 percent of the possible internal joint corners spalled, but approximately 20 percent had more than 15 percent spalled and the poorest performing project had 48.4 percent spalled.

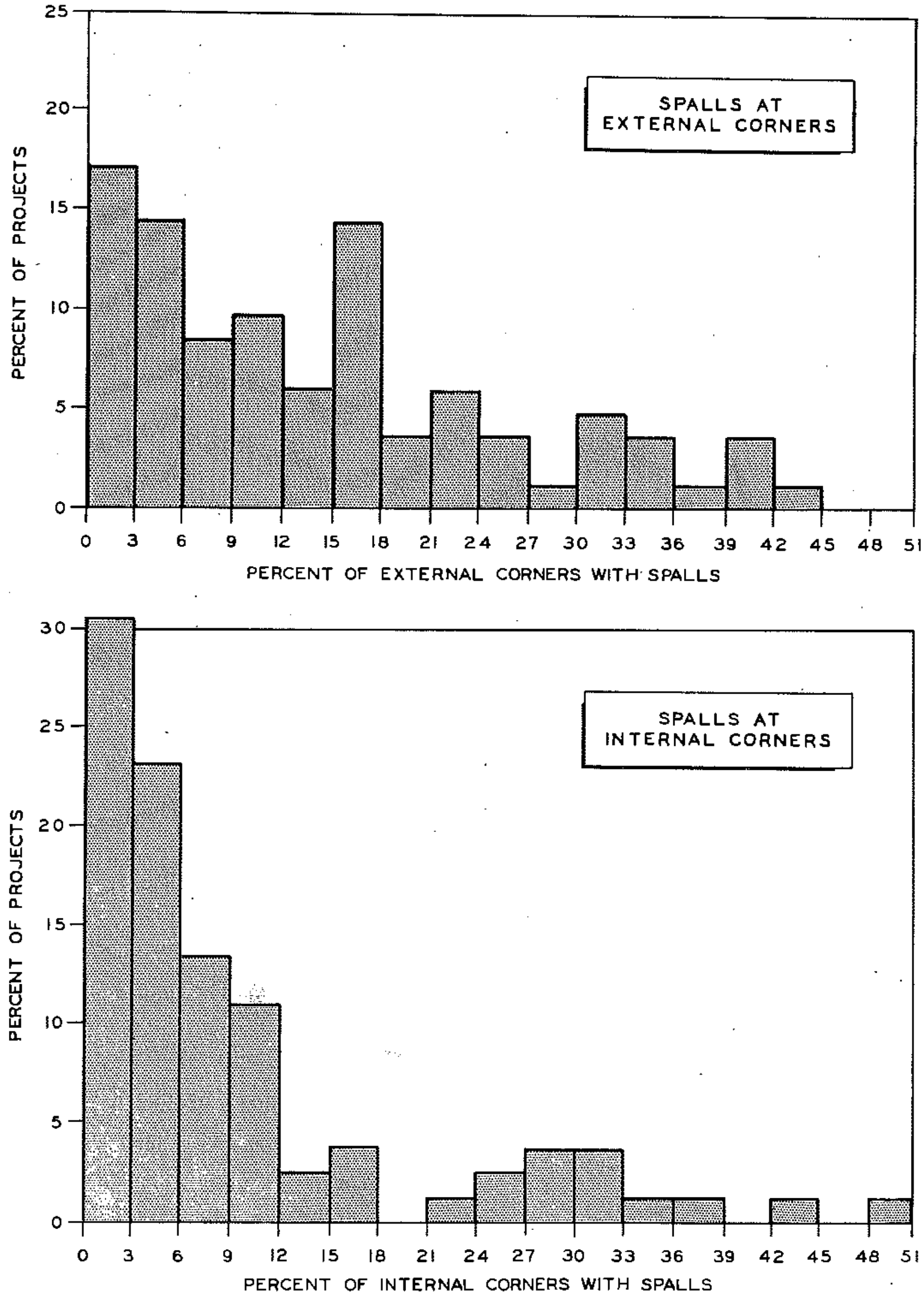


Figure 8. Histograms of corner spalls for 82 two-lane pavement projects; 10 years of service.

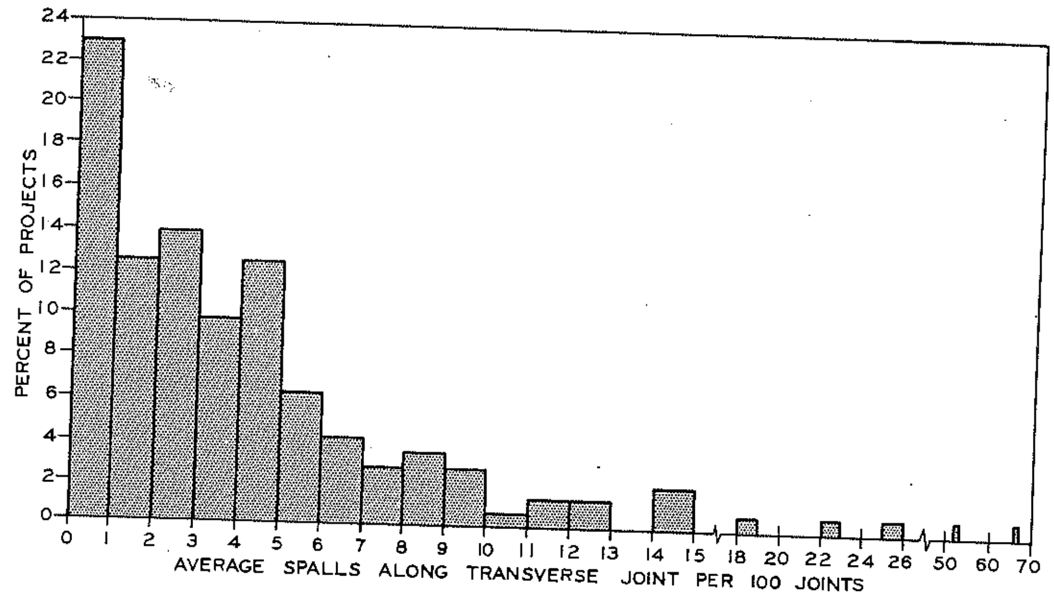


Figure 9. Histograms of average number of spalls along transverse joints for 144 construction projects; 10 years of service.

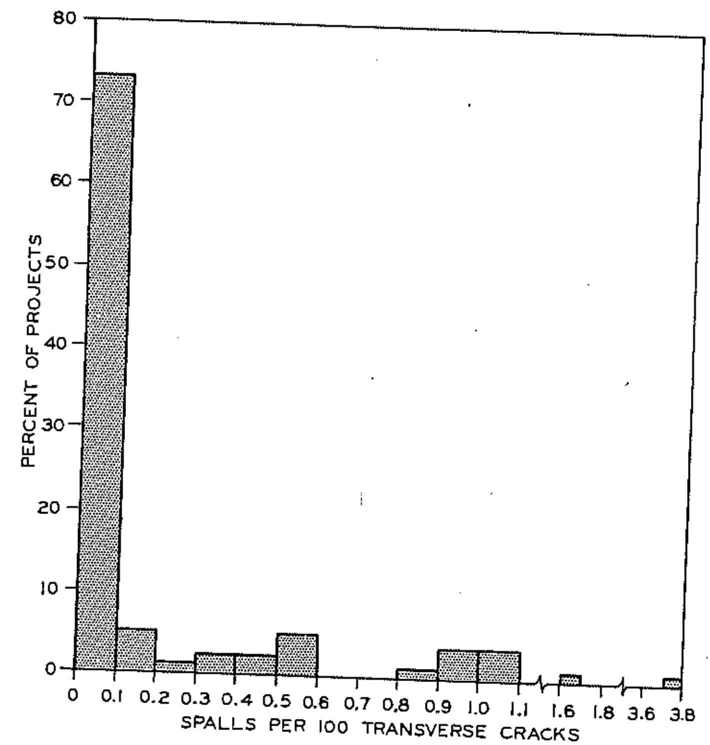


Figure 10. Histogram of spalls at transverse cracks for 82 two-lane projects; 10 years of service.

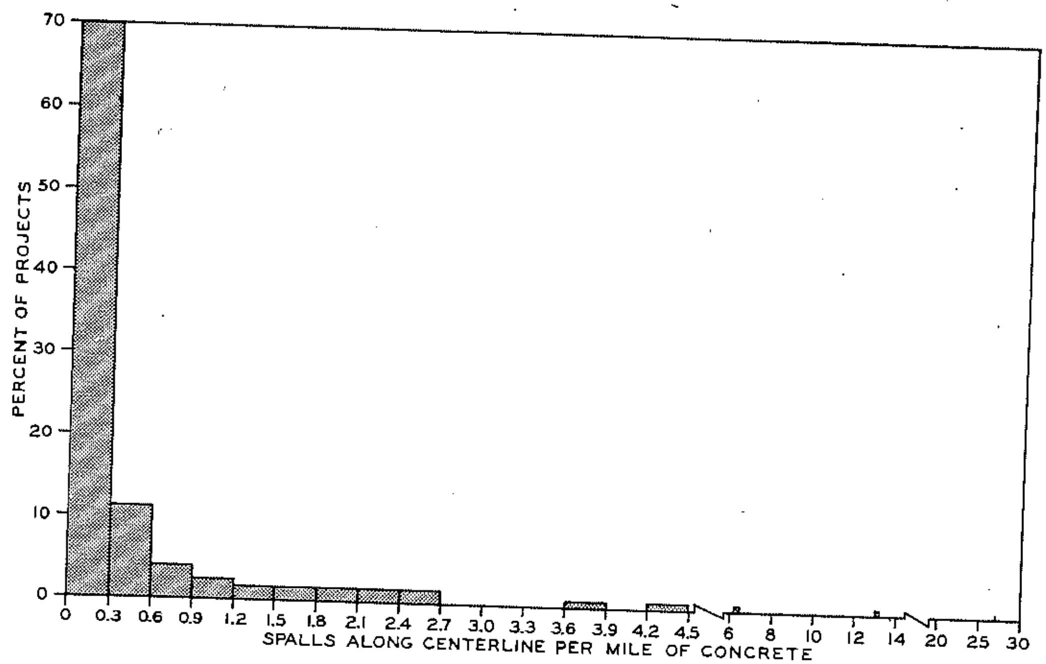


Figure 11. Histogram of spalls along centerline for two-lane and four-lane divided pavement; 10 years of service.

Spalling along the interior portion of transverse joints is shown in Figure 9. The distribution is again extreme, from 22.9 percent of the 144 projects with an average of less than one spall along the interior portion per 100 transverse joints to an average of 70 such spalls per 100 transverse joints or nearly one for every joint. The median performance was 3.11 interior spalls along the joint per 100 transverse joints. Spalling along transverse cracks appears to be no serious problem for most projects, with 73 percent of the projects having no spalling at transverse cracks and 98 percent of the projects have on the average one or less spalls per 100 transverse cracks (Fig. 10). The poorest performing project had an average of 3.8 spalls along transverse cracks per 100 cracks.

Spalling along the centerline joint is indicated by the frequency distribution in Figure 11. Over sixty percent of the projects had no spalling of this type and 88 percent had less than one such spall per mile of pavement. The poorest performing project had 27 spalls per mile of pavement or one approximately every 200 ft.

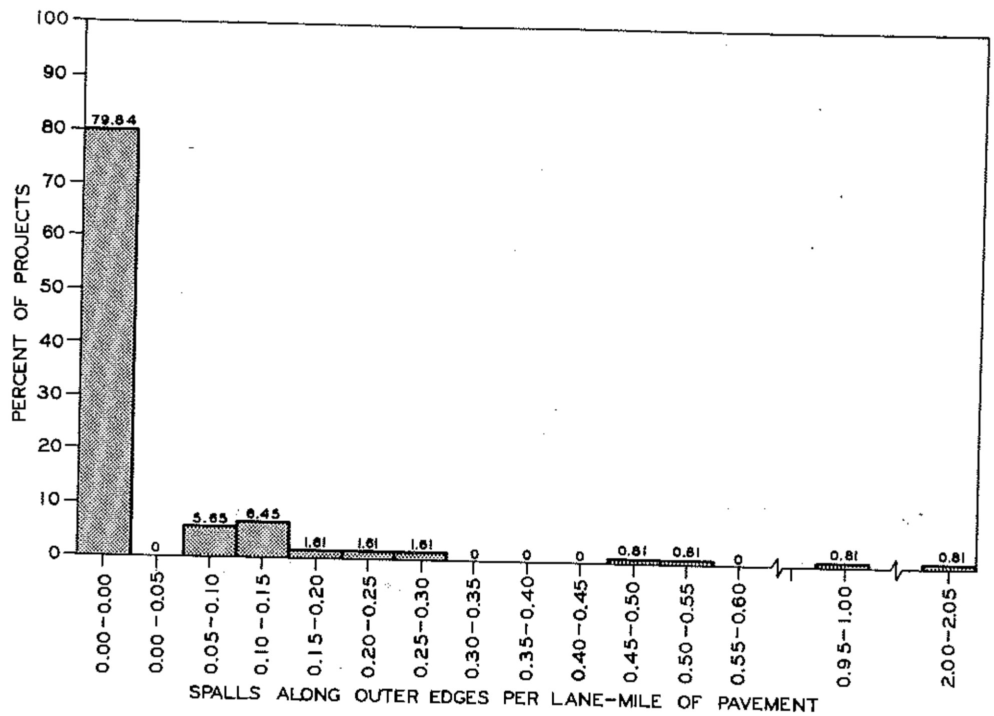


Figure 12. Histogram of spalls along the outer edge of slab for two-lane and four-lane divided highway construction projects; 10 years of service.

Spalling of the longitudinal edge of the pavement is shown in Figure 12. Approximately 80 percent of the projects had no spalling of this type and only one project of 124 had an average of more than one such spall per lane-mile of pavement. This project had approximately two such spalls per lane-mile.

The frequency of spalling of the interior of the pavement slab surface is shown in Figure 13. Seventy-seven percent of the projects had no spalling of this type and only one project had more than one spall per lane-mile. The poorest performing project had 2.9 spalls per lane-mile.

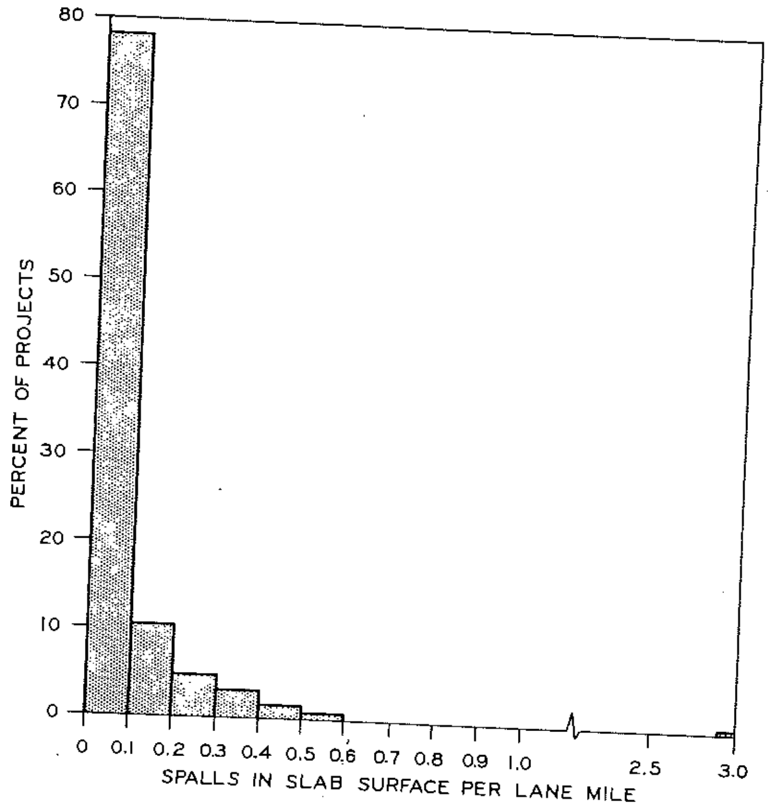


Figure 13. Histogram of spalls in slab surface per lane mile for 124 construction projects; 10 years of service.

5. Joint Blowups

A study of the number of pavement blowups in postwar roadways is of interest since postwar pavements in Michigan were the first to omit expansion joints except at special locations such as bridge approaches, street intersections, railroad crossings, etc. Further, if the pavements were placed prior to April 15 or after September 15, expansion joints were placed at intervals of 396 ft, or every fourth joint, in place of a contraction joint.

For a large part of the pavement projects under study no expansion space was provided. During the late spring, the moist subgrade conditions, along with occasional warm temperatures, does lead to pavement blowups. As these are reported Research Laboratory personnel attempt to visit the site and determine if some construction irregularity, such as improperly centered or misaligned dowels, etc., might have contributed to the blowup at this particular location. A study of two-lane and four-lane divided highways indicated that 63 out of 98 (64 percent of the projects) had no joint blowups at the end of ten years of service. A frequency distribution of blowups is shown in Figure 14. While the largest portion of the projects performed well in this respect, six projects had five percent of the joints blown by the end of ten years of service. The ten poorest projects (10.2 percent) had 62 percent of the pavement joint blowups. This extreme spread in performance has been studied in detail later in the report and causally associated with various material variables.

6. Mud-Jacking

Mud-jacking is required when the pavement settles due to non-uniformity and lack of compaction of the subgrade or subbase prior to the paving operation. Usually the need for mud-jacking is apparent after small amounts of traffic and, therefore, this corrective measure is generally applied early in the service life of the pavement. Two-lane and four-lane, divided and undivided, construction projects (a total of 144) with ten years of service were studied. Of this total 113, or 78 percent, required no mud-jacking. The frequency distribution of mud-jacking as a percent of the original pavement is illustrated in Figure 15. In addition to the 78 percent with no mud-jacking, another 16 projects (11.1 percent) had less than 1 percent of the pavement surface mud-jacked. Seven projects or 4.9 percent had 2 percent or more of the pavement surface that had been mud-jacked and the poorest performing project required 22.8 percent of the pavement surface to be mud-jacked.

7. Patching

After ten years of service, 66 of the 146 projects (45 percent) had no patching and 66 percent had less than 50 sq ft per lane-mile of patched pavement (Fig. 16). Two projects had more than 400 sq ft per lane-mile, and the worst performing project in this category had more than three times as much patching as the next highest project or 1,320 sq ft per lane-mile of pavement (approximately 2.3 percent of the pavement surface).

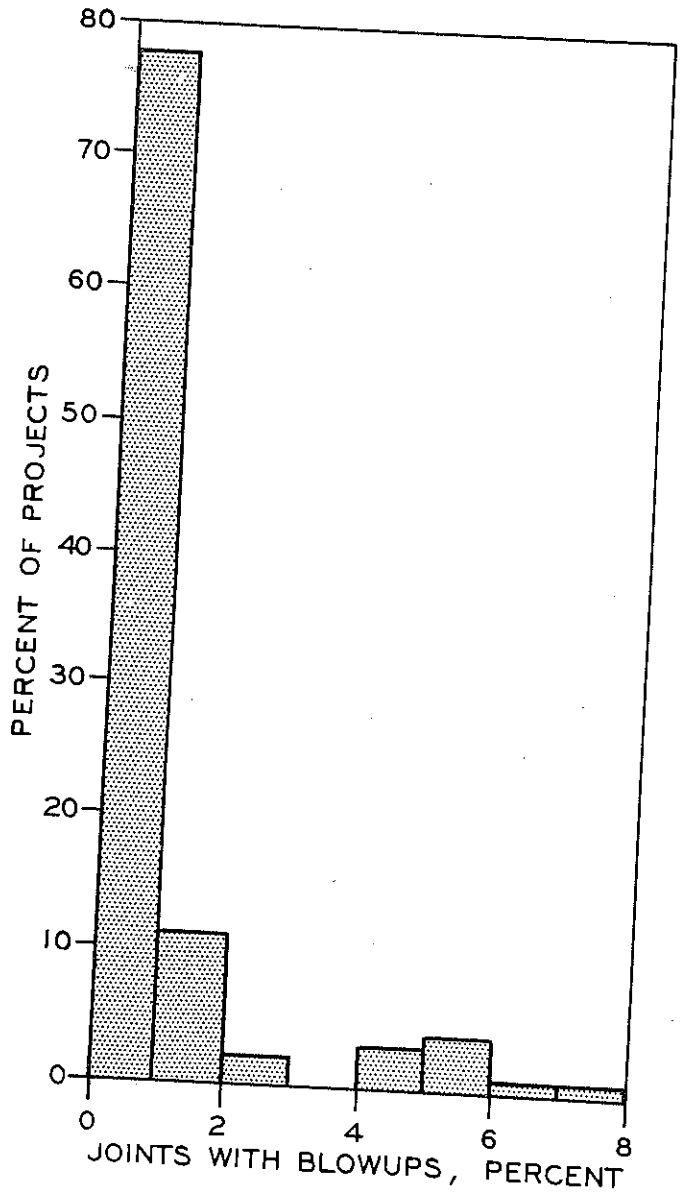


Figure 14. Histogram of joint blowups for 98 projects; 10 years of service.

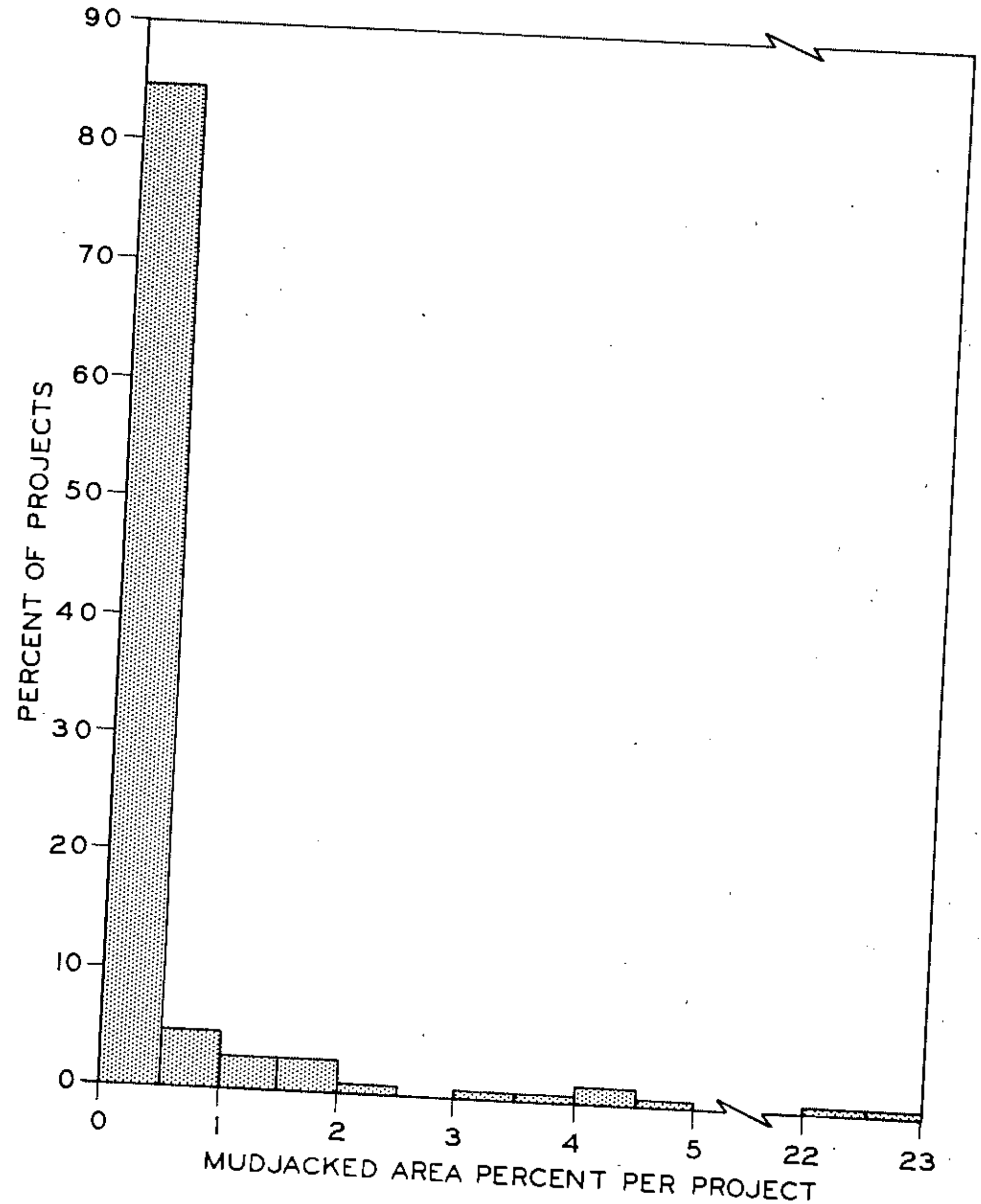


Figure 15. Histogram of percent of pavement requiring mudjacking (144 projects); 10 years of service.

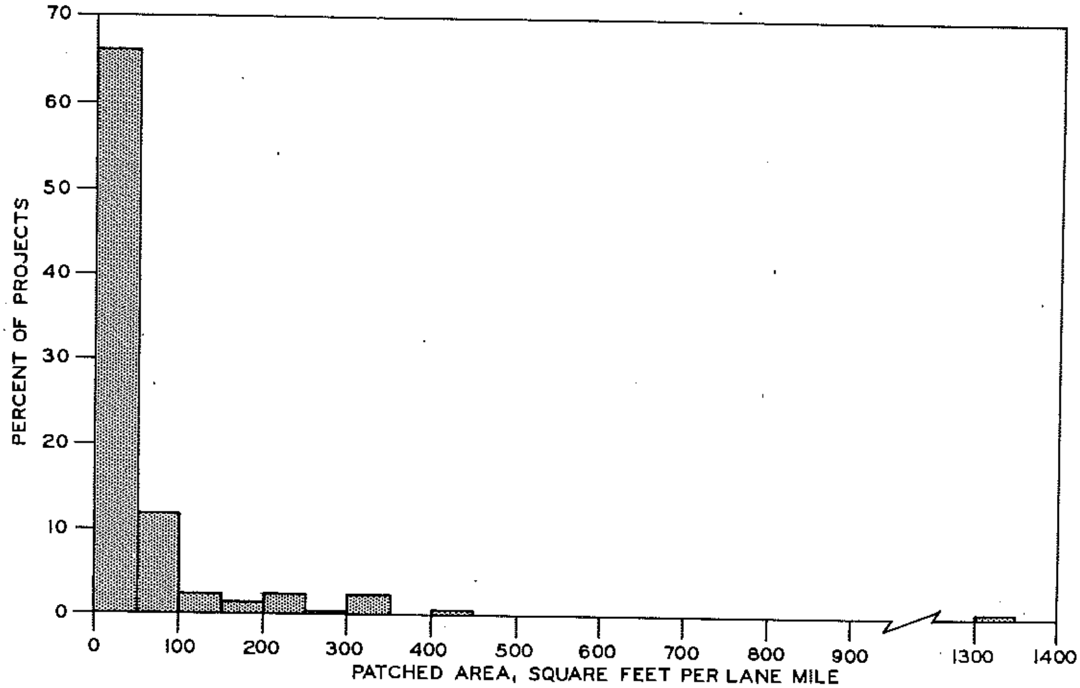


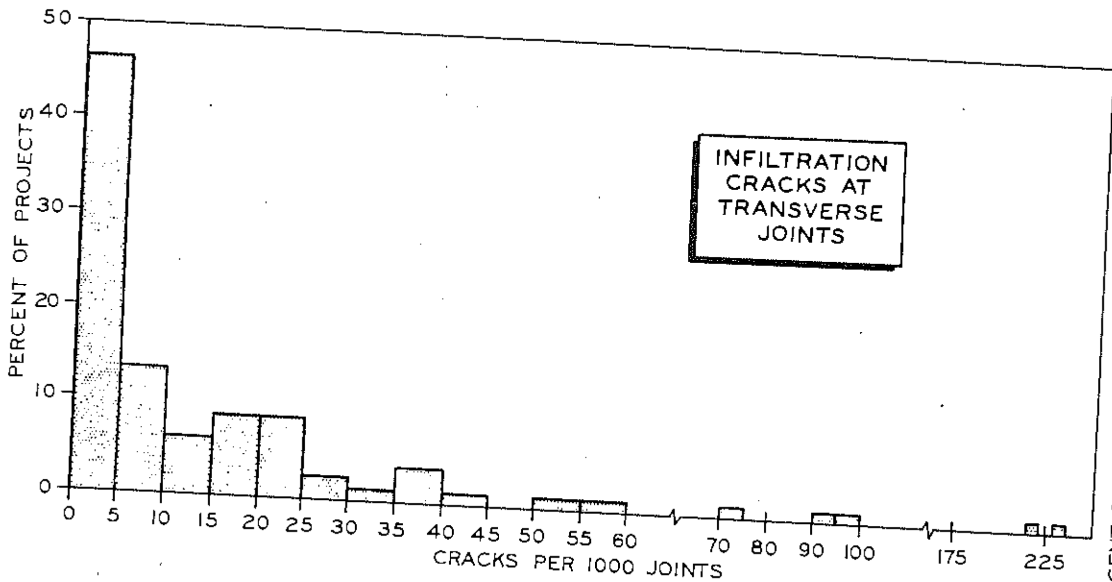
Figure 16. Histogram of patched area for 144 construction projects; 10 years of service.

8. Infiltration Cracks

Infiltration cracks at joints or cracks are short cracks following a course approximately parallel to the centerline and starting from a transverse joint or transverse crack. Figure A4, Appendix I illustrates this type of cracking. Frequency distributions of these two types of cracking are given in Figure 17. For the 82 two-lane projects with ten years of service, 23 had no infiltration cracks at joints and 66 had no infiltration cracks at transverse cracks. At the other extreme, the poorest performance was obtained on two projects where there were 218 and 231 infiltration cracks per 1,000 joints. Infiltration cracks at transverse cracks were not as severe; the two poorest performing projects having 0.42 and 0.47 cracks per lane-mile of pavement.

Fifteen Year Service

Fifty postwar projects had fifteen-year service records. These 50 projects are part of the 144 projects where performance was analyzed on the basis of ten years of service.



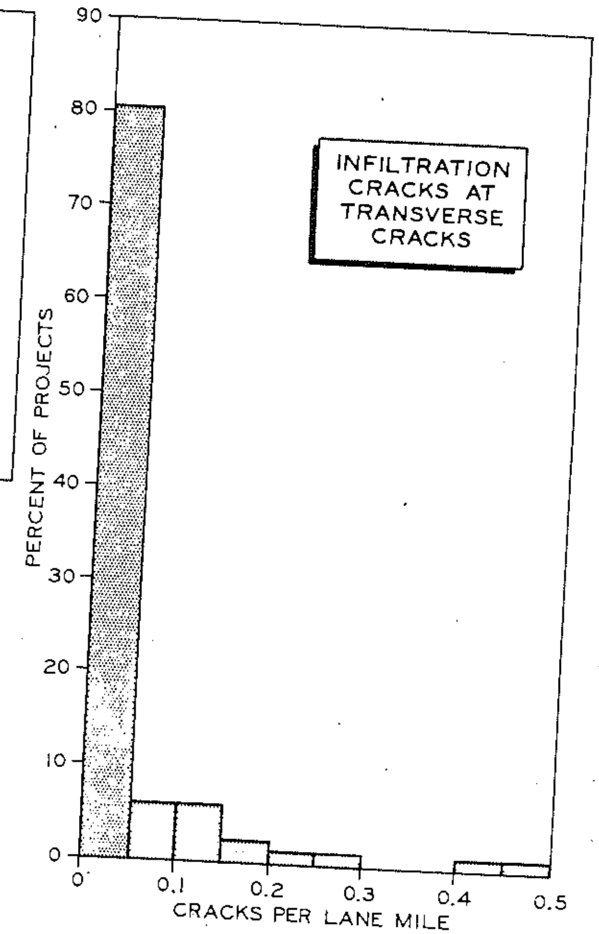
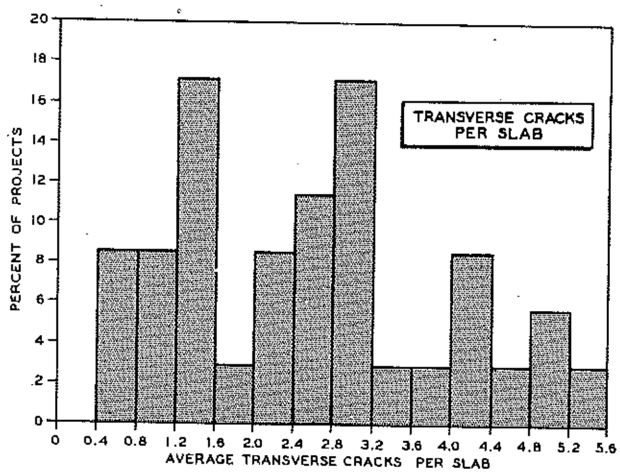


Figure 17. Infiltration cracking at transverse joints and transverse cracks in 82 two-lane projects; 10 years of service.

1. Transverse Cracking

Figure 18 shows the frequency distribution of transverse cracking. Median performance was 2.49 cracks per slab. This compares with 1.50 cracks per slab for projects with ten years of service. The four best projects averaged 0.77 cracks per slab, while the four poorest projects averaged 5.0 cracks per slab.

The frequency distribution of projects with various percentages of uncracked slabs is also shown in Figure 18. The median project had 19.1 percent of the slabs uncracked at the end of fifteen years; the worst performing projects had all slabs cracked and the best performing project had 67.5 percent of the slabs uncracked after fifteen years of service.



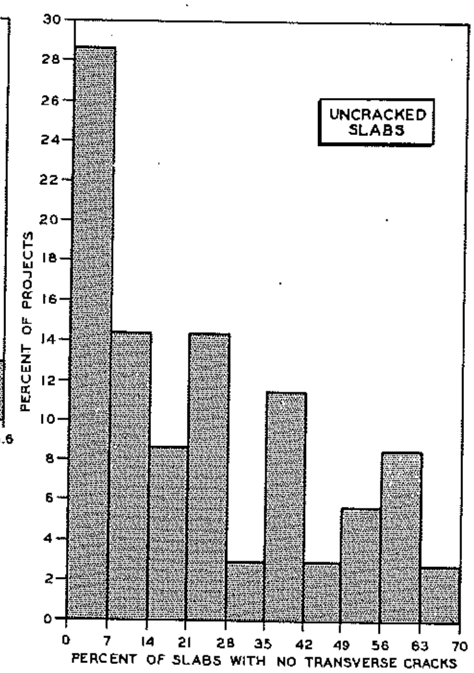


Figure 18. Histograms of transverse cracking for 35 two-lane pavement projects; 15 years of service.

2. Longitudinal Cracking

Variations in the amount of longitudinal cracking for the 44 construction projects are shown in Figure 19. Seven projects (15.9 percent) had no longitudinal cracking, the median performing project had 11.5 ft of longitudinal cracking per mile of pavement and the two performing the poorest had 133 and 136 ft per mile, or approximately 2.5 percent.

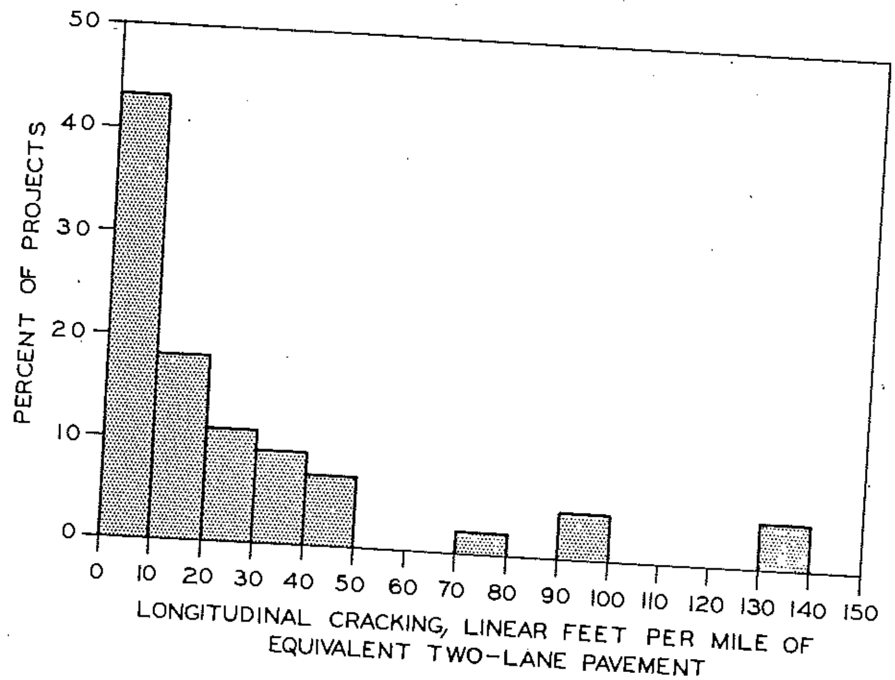


Figure 19. Histogram of length of longitudinal cracking for 44 projects; 15 years of service.

3. Corner Breaks

The distribution of external and internal corner breaks among 35 two-lane construction projects is shown in Figure 20. In five projects (14 percent) there were no external corner breaks and in 12 projects (34 percent) there were no internal corner breaks. Median performance was 0.46 percent of the external corners with corner breaks and 0.18 percent of the internal corners. The four poorest performing projects had 2.14, 2.16, 23.39, and 42.94 percent of the external corners broken. With respect to internal corner breaks, the four poorest projects had 0.88, 1.13, 4.42, and 13.45 percent of the corners broken. It is hardly a coincidence that the same project was the poorest with respect to both types of corner breaks. This project was intensively studied about ten years ago in an attempt to determine the cause of this poor performance.

The distribution of external and internal corner breaks at transverse cracks of pavement with fifteen years of service is shown in Figure 21. Seven of 35 projects had no corner breaks of this type and the median project had 0.27 corner breaks per 100 transverse cracks. The four poorest projects had 1.20, 1.32, 1.34, and 1.70 corner breaks per 100 transverse cracks.

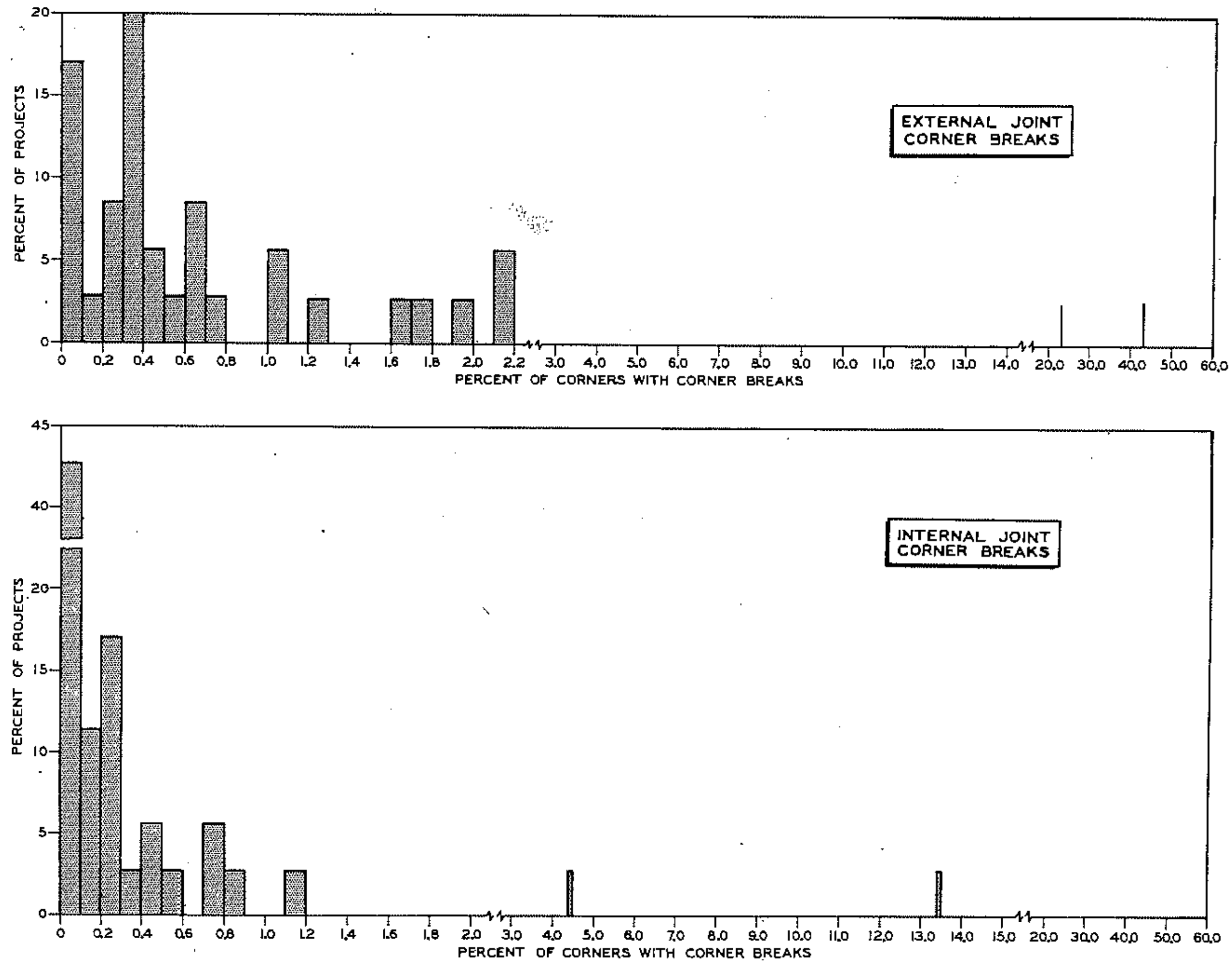


Figure 20. Histograms of corner breaks for two-lane pavement projects; 15 years of service.

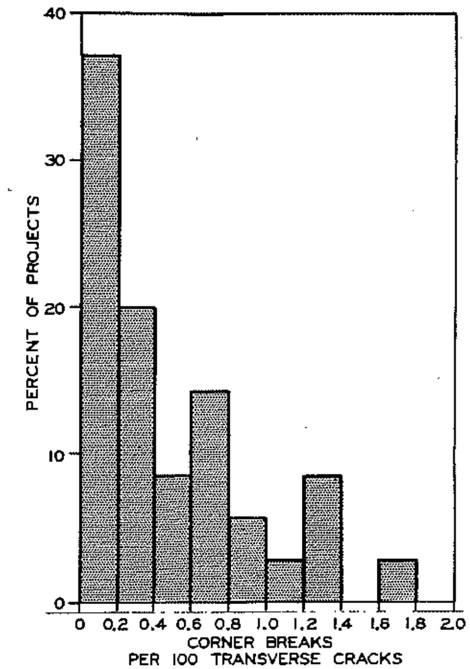


Figure 21. Histogram of corner breaks at transverse cracks for 35 pavement projects; 15 years of service.

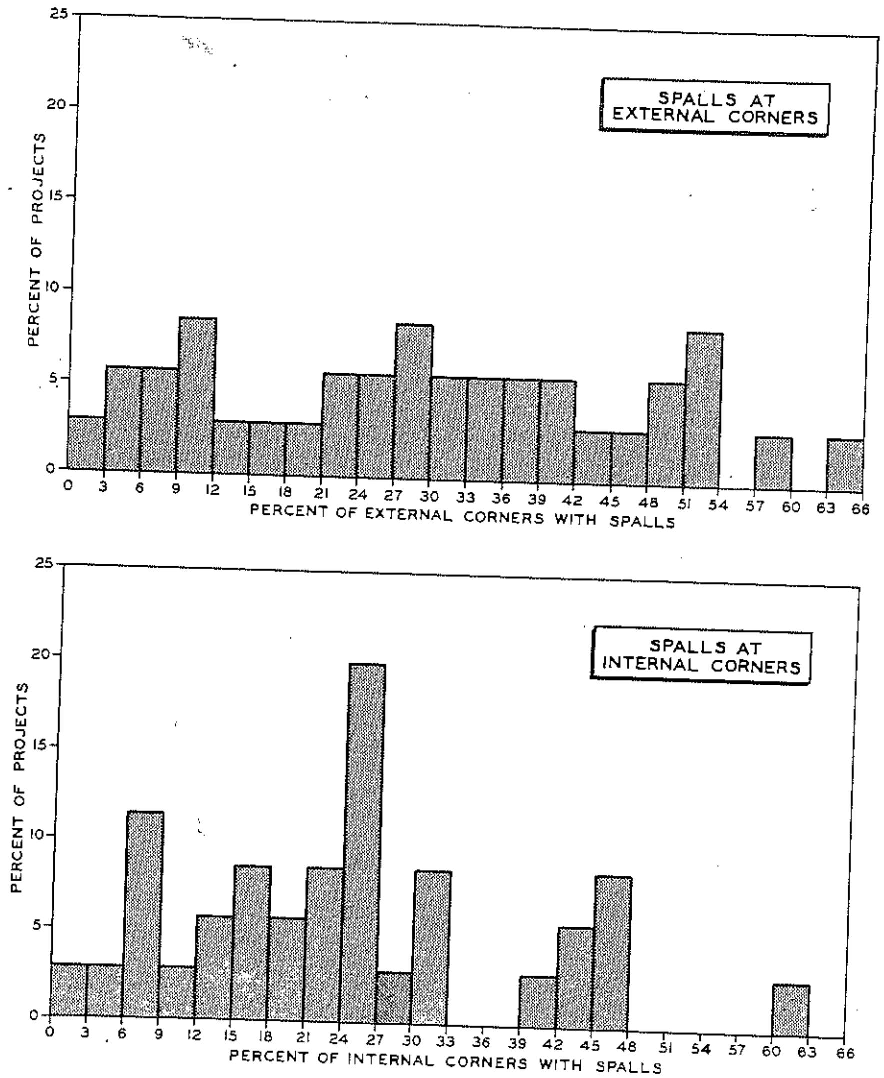


Figure 22. Histograms of corner spalls for 35 two-lane pavement projects; 15 years of service.

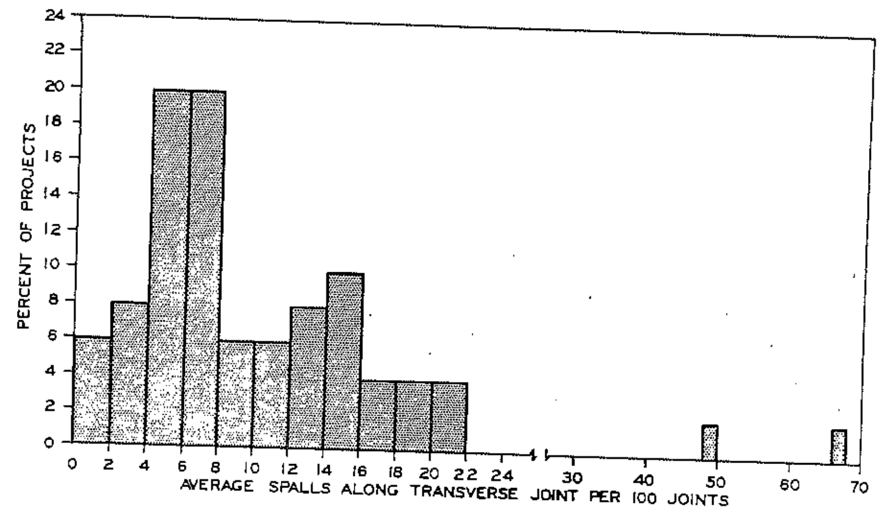


Figure 23. Histogram of average total spalls along transverse joints for 35 construction projects; 15 years of service.

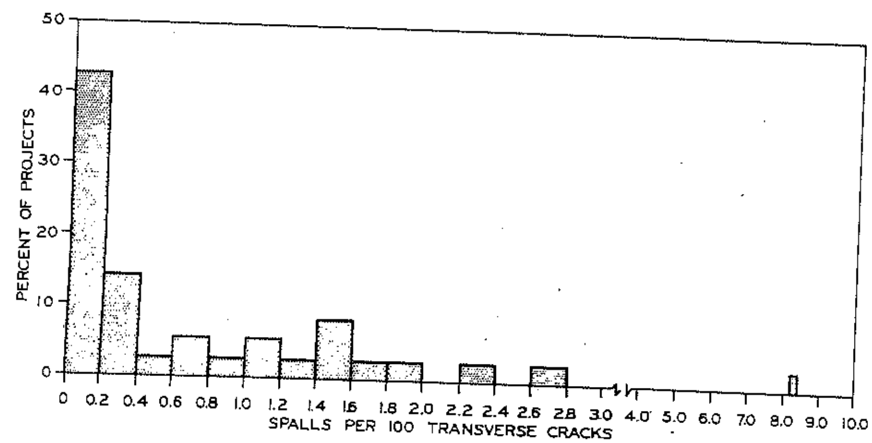


Figure 24. Histogram of spalls at transverse cracks for 35 two-lane projects: 15 years of service.

4. Spalls

The severity of external and internal corner spalling is shown in Figure 22. Median performance for the 35 projects with fifteen years of service was 29.2 percent of the external corners with spalls and 18.7 percent of the internal corners. The four poorest projects in each category had 53.3, 53.6, 58.6, 65.2 percent of the external corners with spalls and 46.9, 47.2, 47.9, and 60.1 percent of the internal corners with spalling. In comparing the frequency distribution of corner spalls for ten (Fig. 8) and fifteen years of service it is apparent that a large increase in this type of deterioration takes place in this period. This aspect of pavement performance warrants design improvement to reduce future maintenance. However, one design change introduced in 1964 should markedly reduce the amount of joint spalling. In the 1964 construction season, the Department began using preformed neoprene joint seals in transverse joints. Since that time, annual inspections have shown an absence of soil or stone infiltration into the joint groove. The hot-pour rubber-asphalt seals used before this time became impregnated with solid materials after a few years of service and thus became incompressible. We feel this was the primary cause of the large amount of joint spalling previously discussed.

Spalling along transverse joints not at the joint corners is illustrated in Figure 23. The four best projects had an average of 1.65 percent of the joints with spalling of this type while the four poorest performers ranged from 20.2 to 66.0 percent of the joints and averaged 38.8 percent. The frequency distribution among projects of spalling along transverse cracks is shown in Figure 24. Ten of the 35 projects had no spalling of this type, while median performance was 0.3 spalls per one hundred cracks. The poorest performing project had 8.2 spalls per one hundred cracks.

Spalling along the longitudinal centerline joint ranged from none for 22 projects out of 44 to 6.3 spalls per mile of pavement for the poorest performing project. The frequency distribution among projects for this type of spalling is shown in Figure 25.

Performance was even better for spalling along the outside edge of the pavement where 31 of the 44 projects had no spalling of this type and the poorest performer had only 0.85 spalls per mile of pavement (Fig. 26). Spalling on the interior of the slab surface was non-existent for 26 of the 44 projects and the maximum on a project was 0.9 spalls per lane-mile of pavement, as illustrated by the frequency distribution in Figure 27.

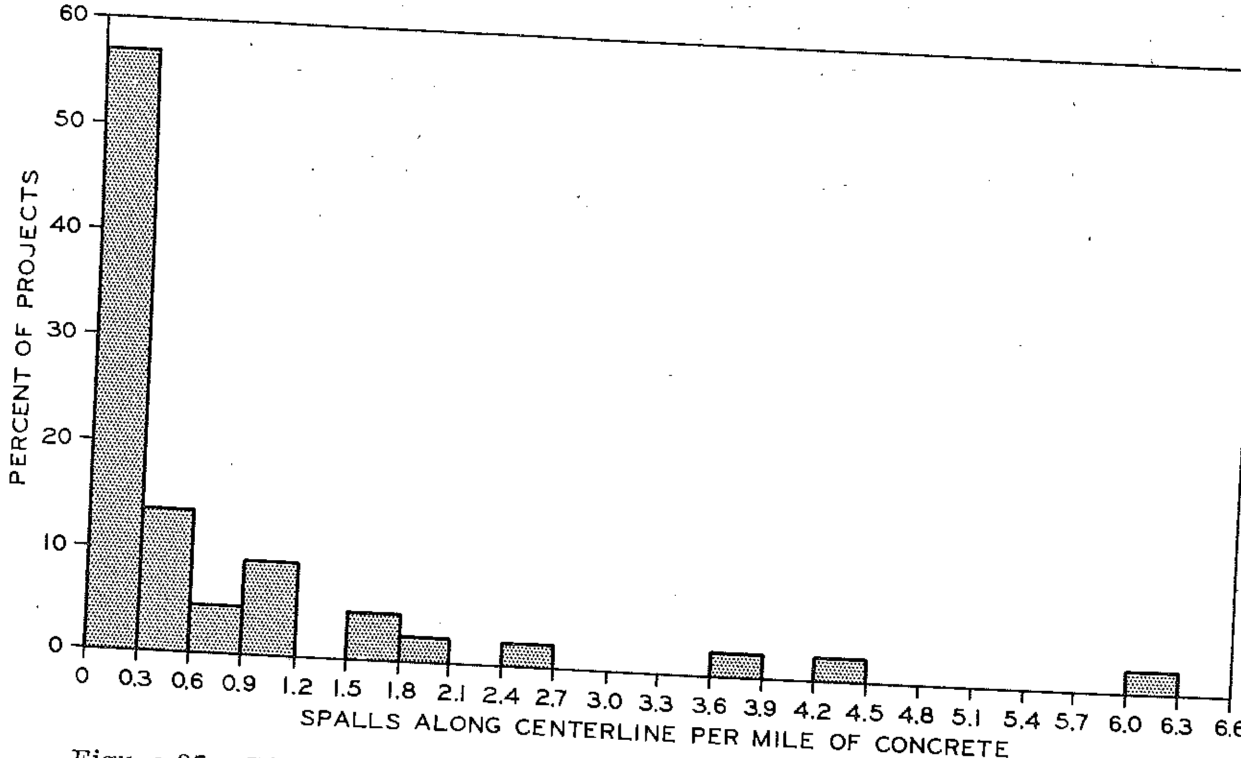


Figure 25. Histogram of spalls along center line for two-lane and four-lane divided pavement; 15 years of service.

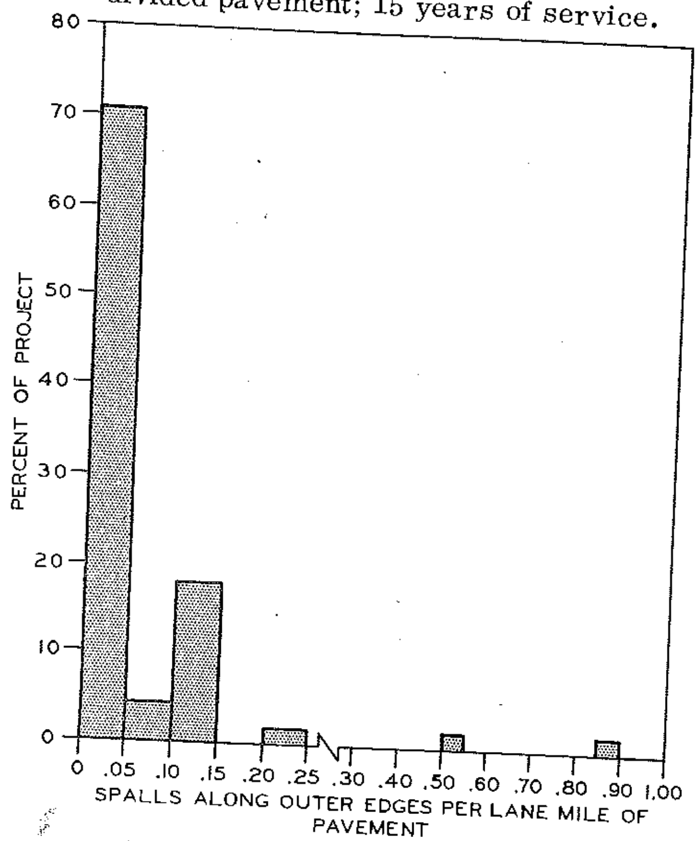


Figure 26. Histogram of spalls along the outer slab edges for two-lane and four-lane divided construction projects; 15 years of service.

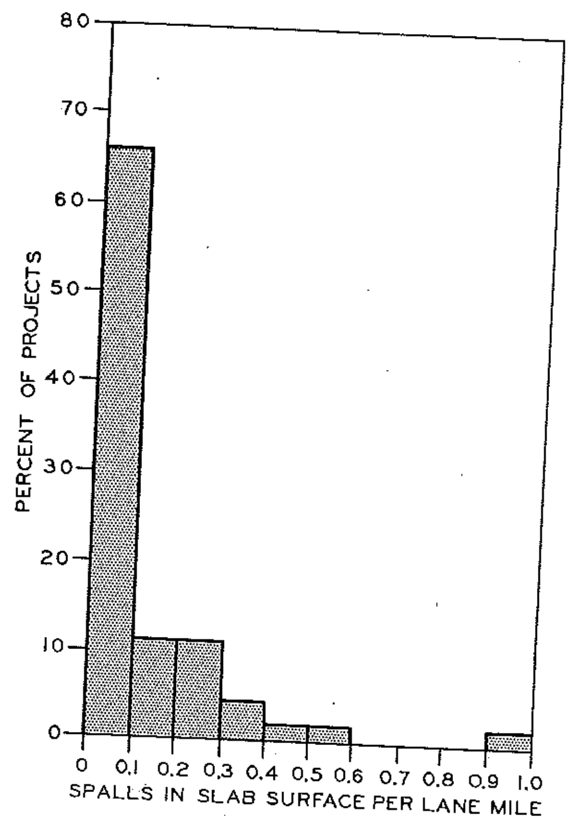


Figure 27. Histogram of spalls in slab surface per lane mile for 44 construction projects; 15 years of service.

In summary, it appears that the most serious type of spalling after fifteen years of service is the spalling of external and internal corners at transverse joints, followed by spalling along transverse joints away from joint corners. If spalling could be controlled at the transverse joint the remaining spalling would be relatively negligible.

5. Joint Blowups

Pavement joint blowups for projects with fifteen years of service are shown in Figure 28. Six of 38 projects had none, the median performance project had 1.4 percent of the joints with blowups and the four poorest projects had 11.5, 17.3, 23.4, and 31.0 percent of their joints with blowups.

6. Mud-Jacking

The prevalence of mud-jacking is shown in Figure 29. Forty-one of 50 projects had no mud-jacking but the four poorest projects in this respect had 2.1, 2.3, 4.7, and 5.0 percent of the pavement surface which had been mud-jacked in order to improve riding quality.

7. Patching

Seven of 50 construction projects had no patched areas after fifteen years of service (Fig. 30). The median performance project had 75 sq ft of patching per lane-mile of pavement (0.13 percent) and the four poorest performing projects in this respect had 830, 910, 1,260, and 1,500 sq ft per lane-mile of pavement or 1.4 to 2.6 percent of the pavement surface (Fig. 30).

8. Infiltration Cracks

The severity of infiltration cracks at joints is shown in Figure 31. Eleven of 38 two-lane projects with fifteen years of service had none of this type of cracking. Median performance was approximately five infiltration cracks per 1,000 joints but the four poorest projects had 18, 36, 47, and 93 such cracks per 1,000 joints. Infiltration cracking at transverse cracks is also shown in Figure 31 as distributed for 35 construction projects. Twenty-one of the projects did not have this type of cracking and the four poorest performing projects had 0.27, 0.43, 0.81, and 2.79 infiltration cracks per lane-mile of pavement.

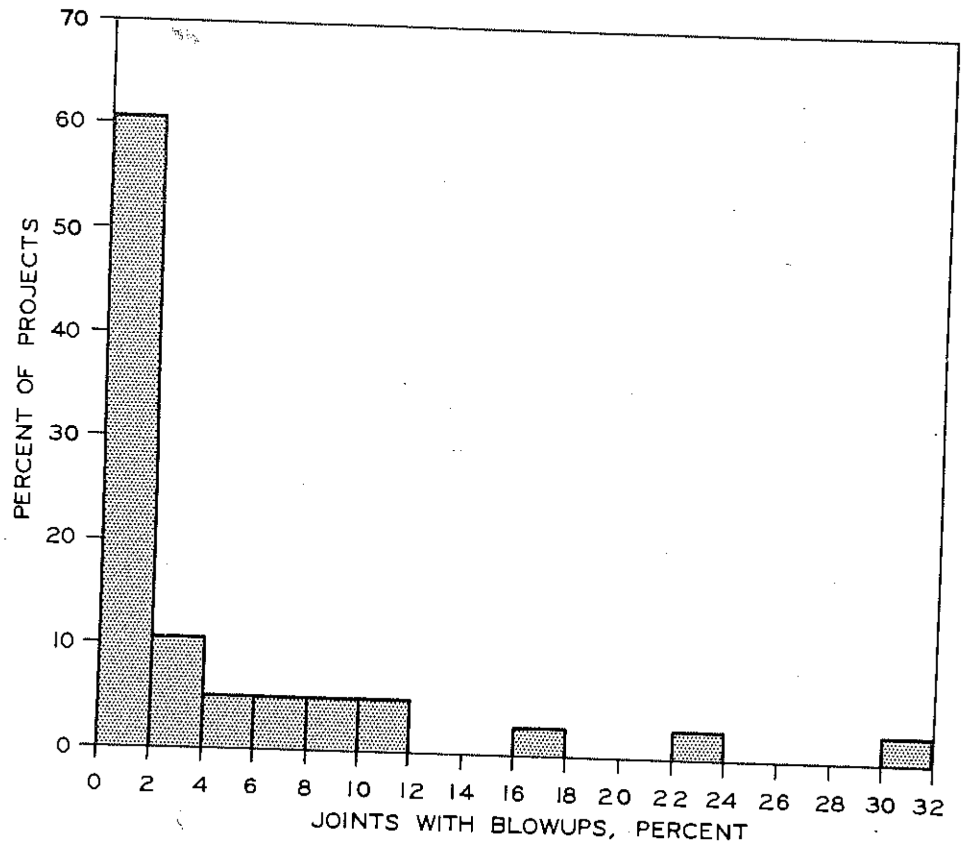


Figure 28. Histogram of joint blow-ups for 38 projects; 15 years of service.

Figure 29. Histogram of percent of pavement requiring mudjacking (50 projects); 15 years of service.

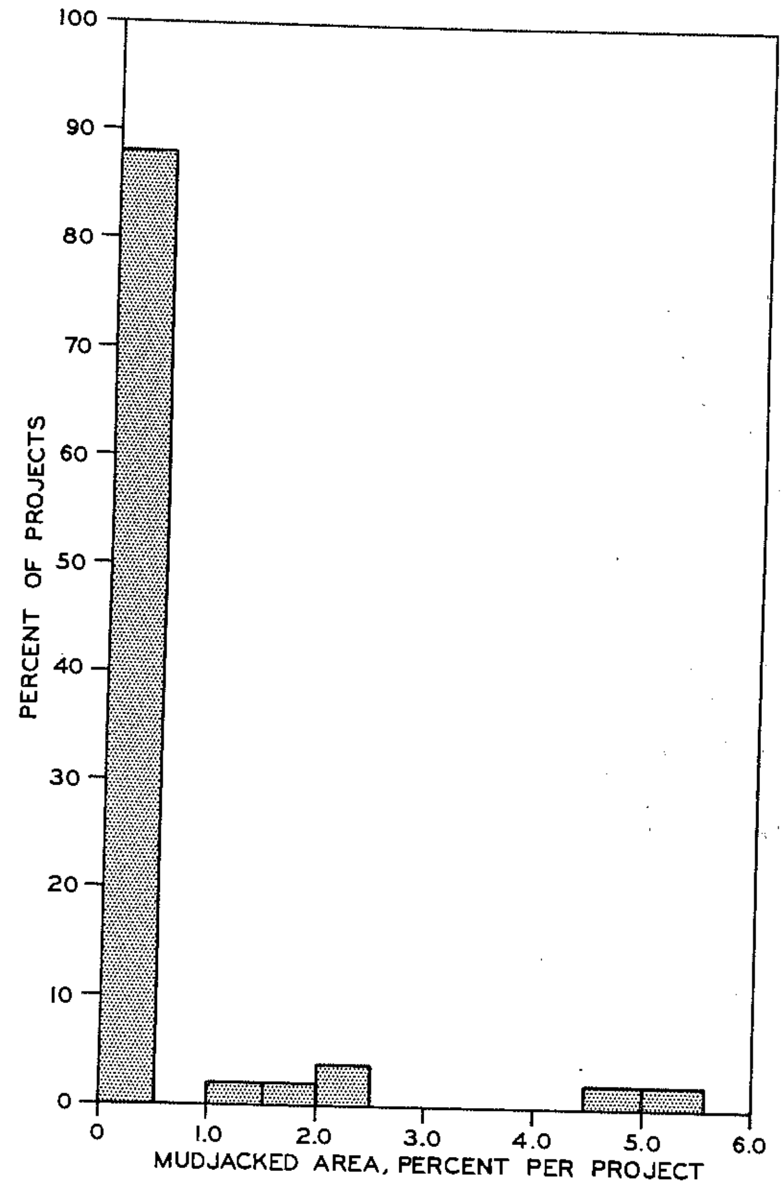
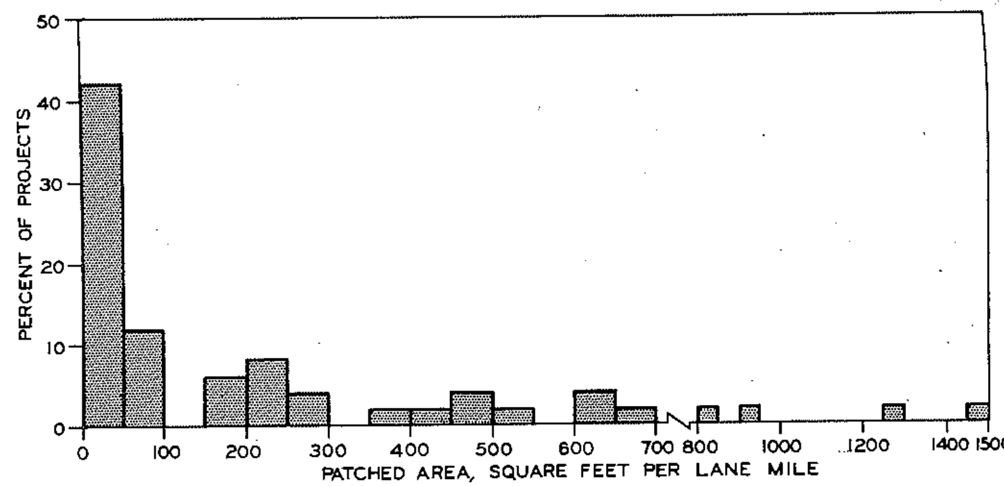


Figure 30. Histogram of patched area for construction projects; 15 years of service.



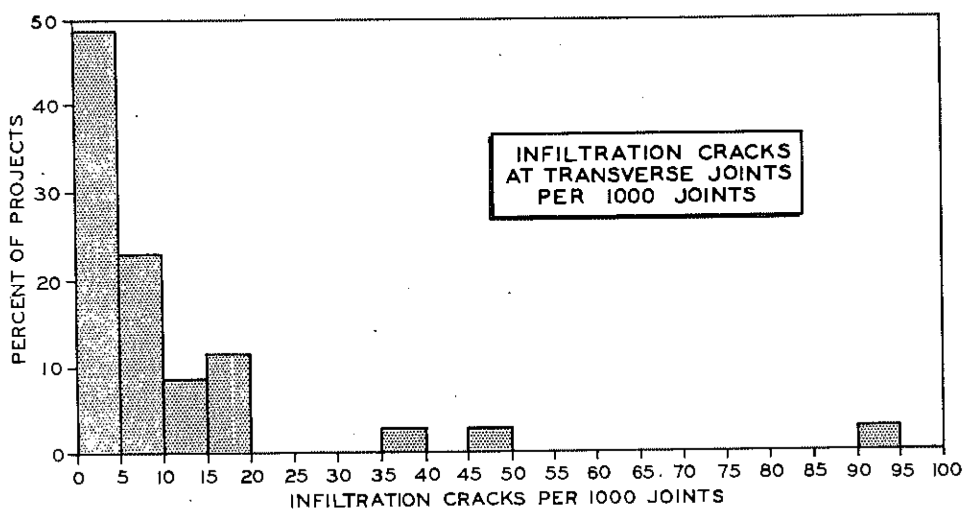
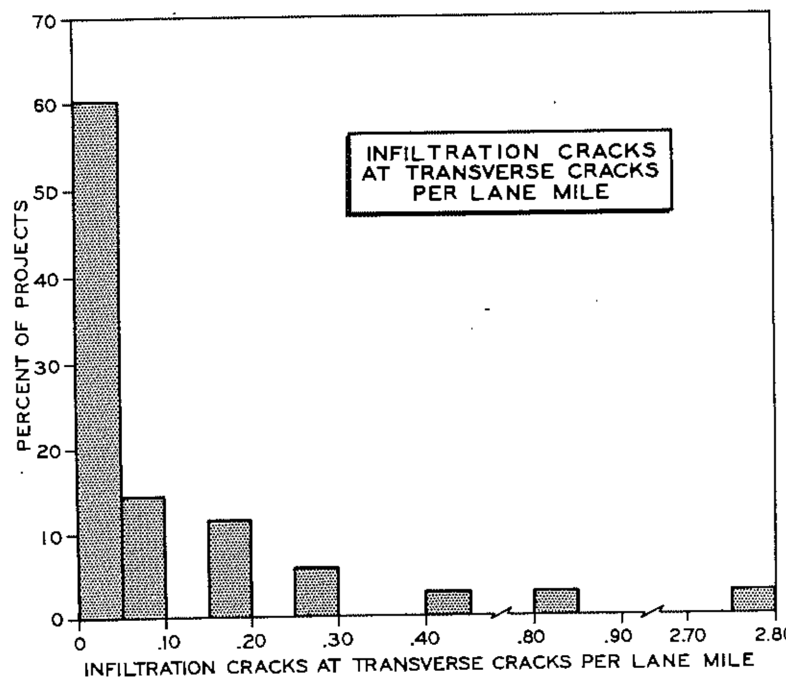


Figure 31. Histogram of infiltration cracks for all two-lane projects; 15 years of service.



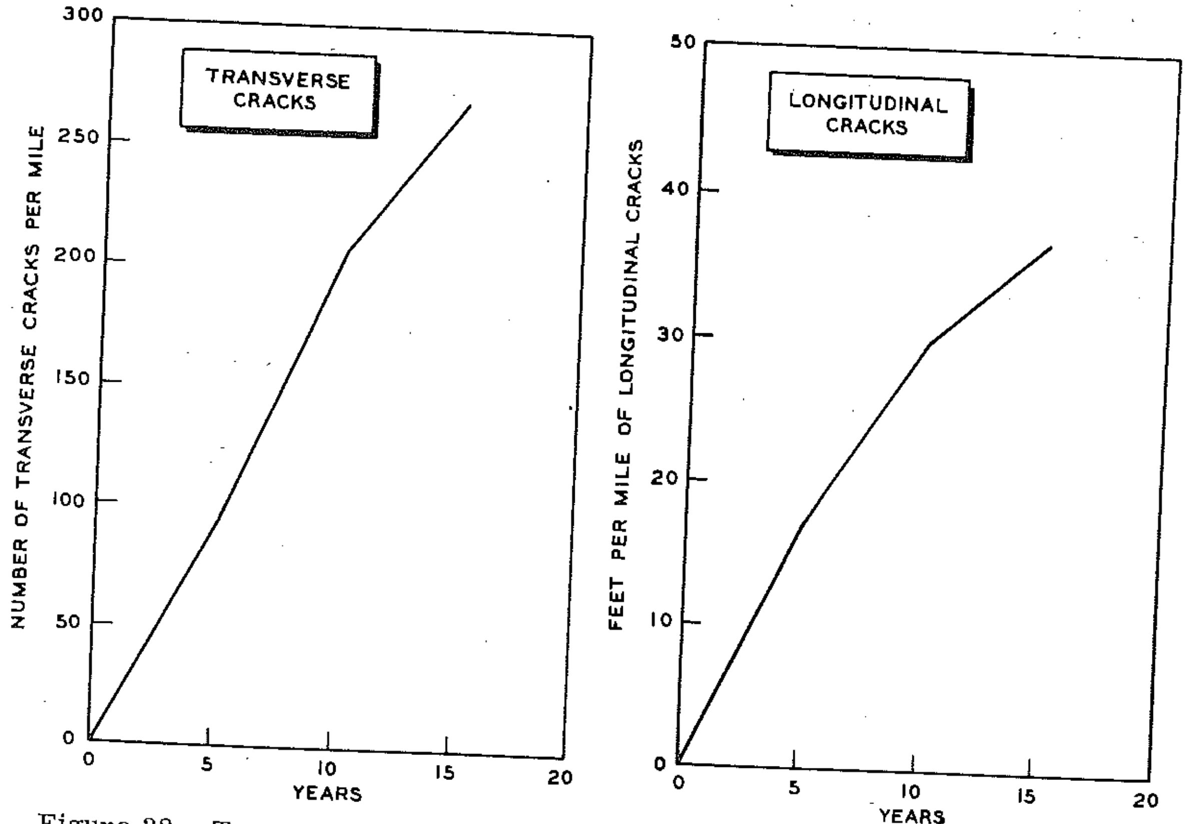


Figure 32. Transverse cracks and longitudinal cracks per mile of two-lane pavement, based on 28 projects with 5, 10, and 15 years of service.

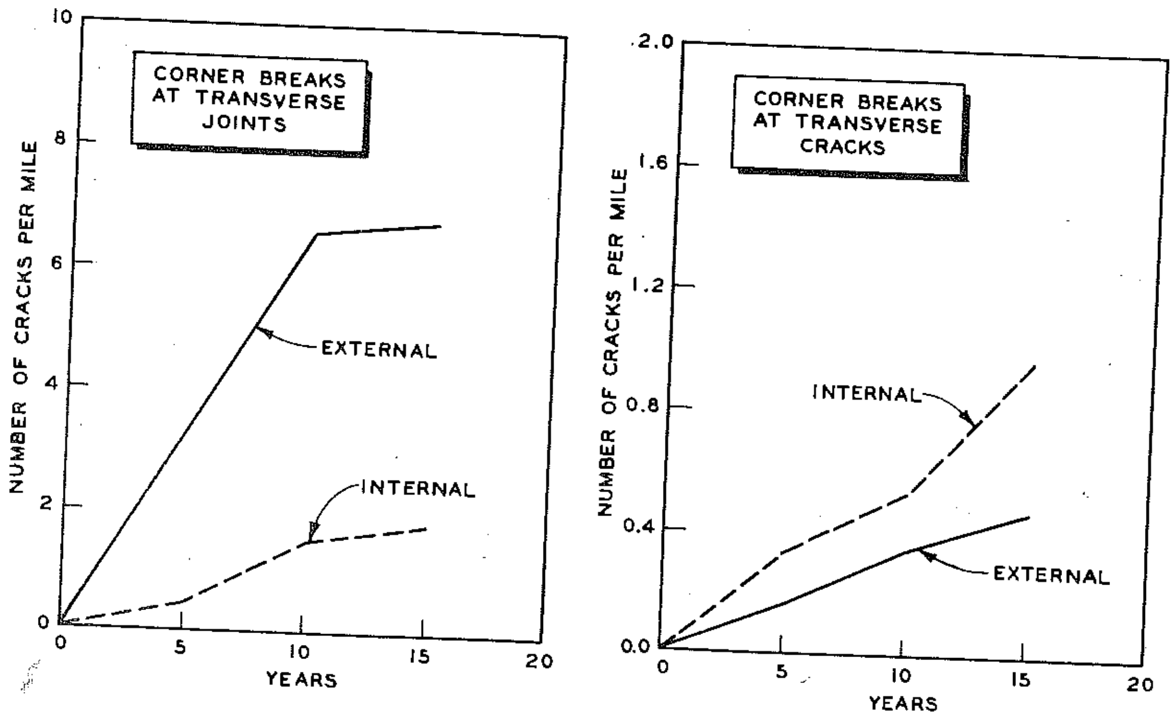


Figure 33. Number of corner breaks per mile of two-lane pavement. This was based on 28 projects, with service checks taken at 5, 10, and 15 years.

Progressive Deterioration

From the condition surveys available for this study it was decided that a study of performance after ten and fifteen years of service would be most rewarding. However, five-year service records were also available for study. In order to determine the rate of change over the fifteen years of service, it appeared most feasible to study the same projects during this span. Figures 32 through 37 illustrate the average progressive deterioration for various types of pavement distress for 28 construction projects built in 1947, 1948, and 1949. Throughout the service period the average number of transverse cracks for the 28 projects increased rather linearly. Although not shown, it is interesting to note that the poorest project with respect to transverse cracking was the same for all three periods of service.

The rate of increase for longitudinal cracking tends to decrease with time. This might be expected since two primary causes of longitudinal cracking, premature traffic loading and subgrade settlement, have their greatest effect at an earlier age. The progressive nature of corner breaks at transverse joints and transverse cracks is shown in Figure 33. Except for external corner breaks during the ten- to fifteen-year period the increase is generally quite linear. In contrast, spalling at transverse joint corners and at transverse crack corners increases at an increasing rate with service life, with very large increases noted in the ten- to fifteen-year service period (Fig. 34). Spalling at other locations also increases very rapidly in the ten- to fifteen-year service period (Fig. 35).

Another form of pavement distress, joint blowups, shows a rapid and progressive increase during the ten- to fifteen-year period (Fig. 36). However, spalling is an earlier distress phenomenon in the life of the pavement and a blowup is a more mature one. During the first five years, only one project out of 28 had any blowups, but by the end of ten years, 11 projects had blowups, and by fifteen years only four projects had no blowups. For those four projects without blowups, the coarse aggregate came from pits with either high 80-100 percent carbonate or very low 0-20 percent carbonate. However, the project which had a blowup by the end of five years was also the worst performer at the end of fifteen years, showing that the character of a bad performing project is indicated early.

Pavement deterioration, resurfacing due to deterioration, and patching are three more pavement distress conditions which progressively become more severe with time as shown in Figure 37.

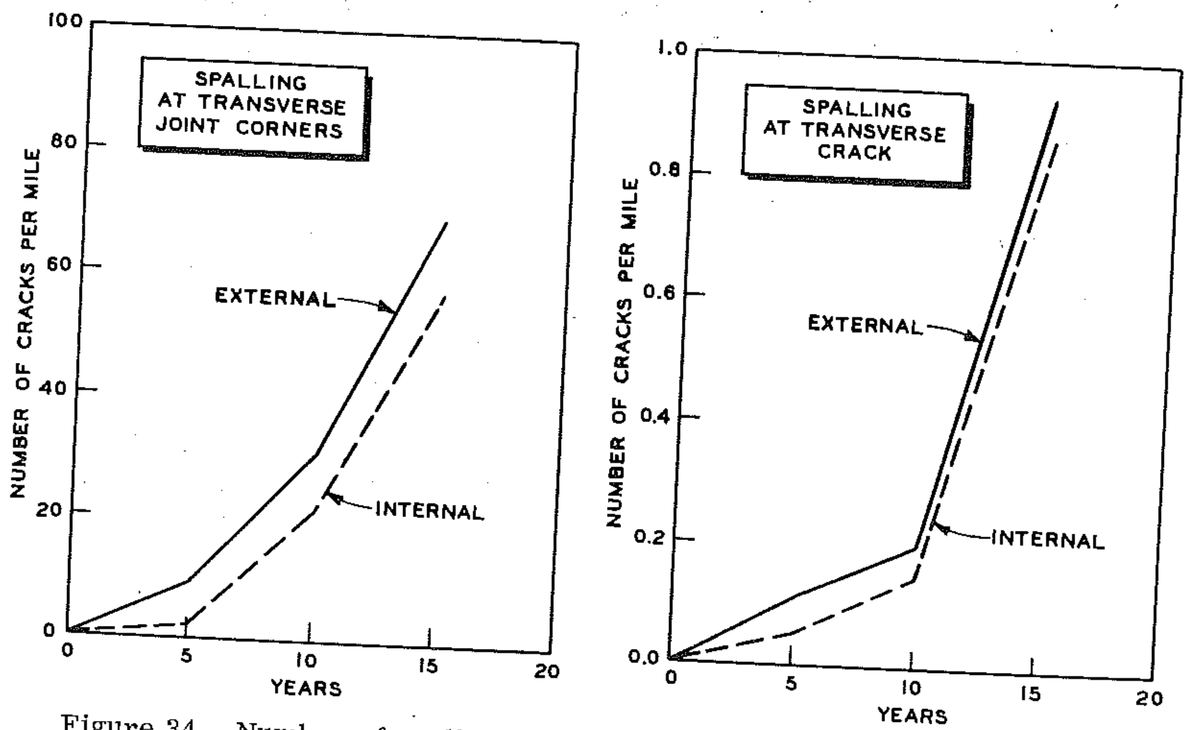


Figure 34. Number of spalls per lane of two-lane pavement, based on 28 projects with service checks at 5, 10, and 15 years.

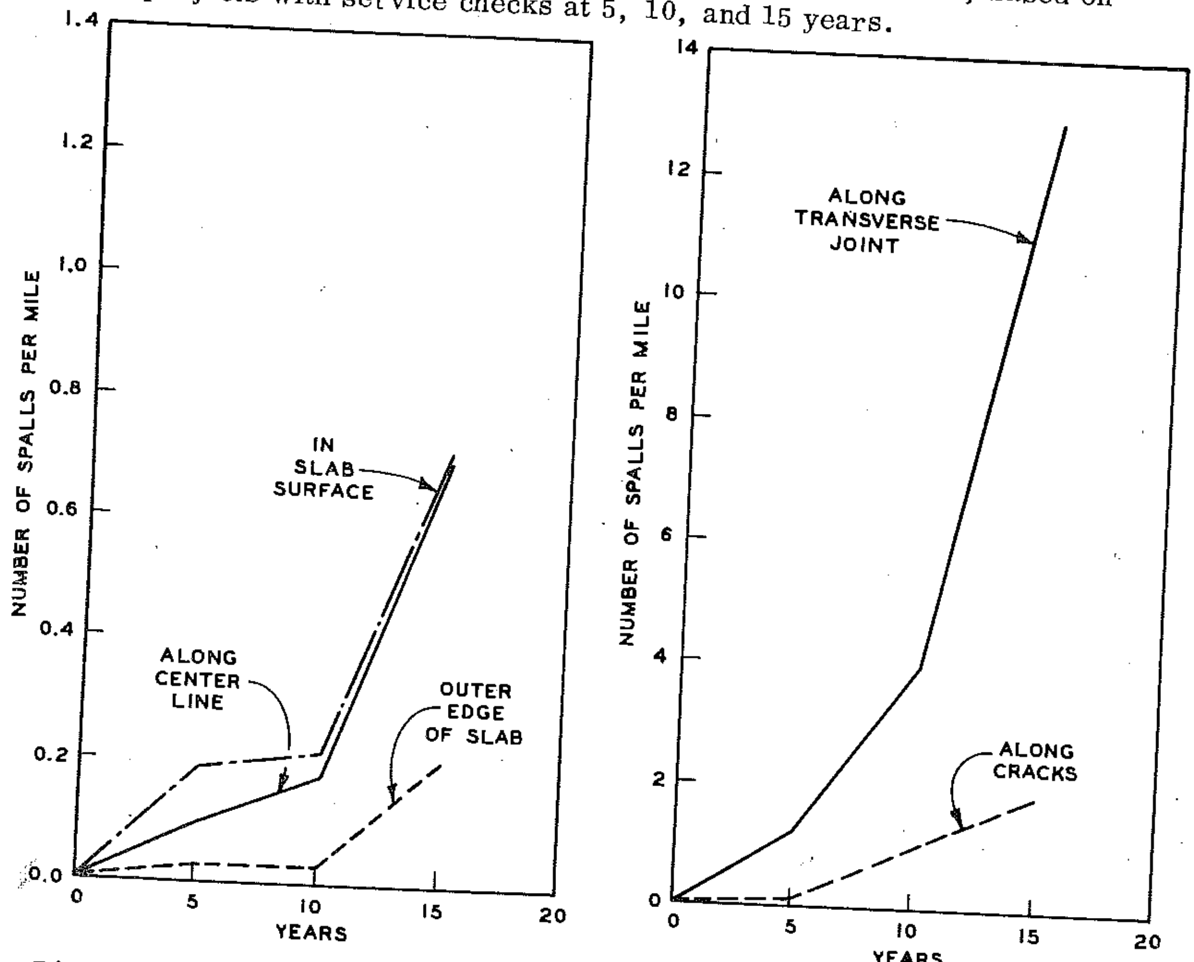


Figure 35. Number of various types of spalling per mile of two-lane pavement, based on 28 projects with service checks at 5, 10, and 15 years.

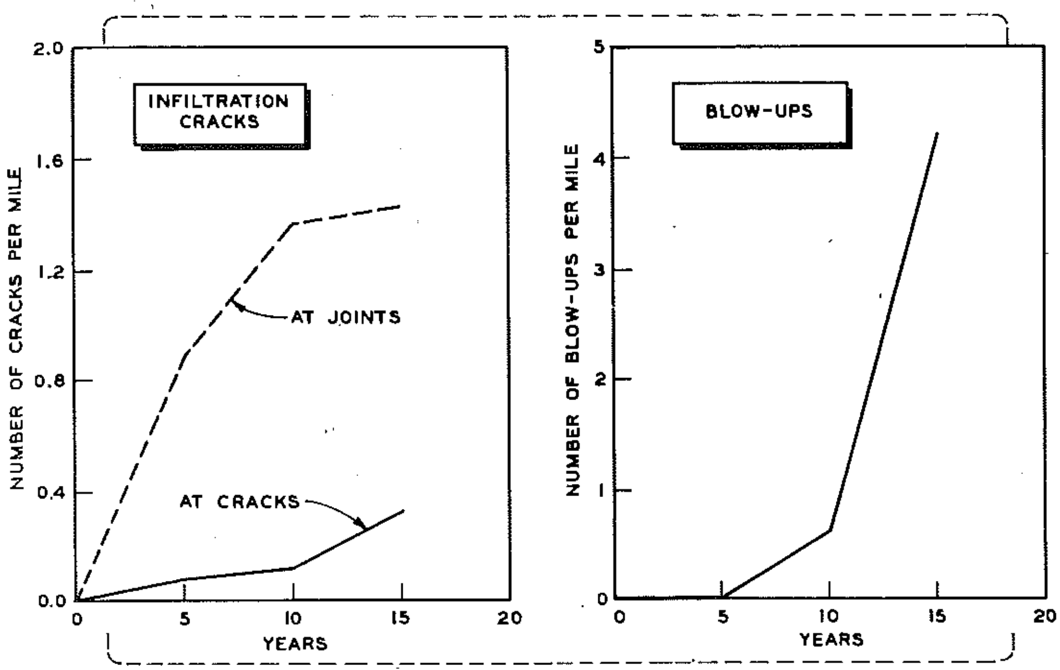


Figure 36. Number of infiltration cracks per mile and number of blowups per mile of two-lane pavement, based on 28 projects after 5, 10, and 15 years of service.

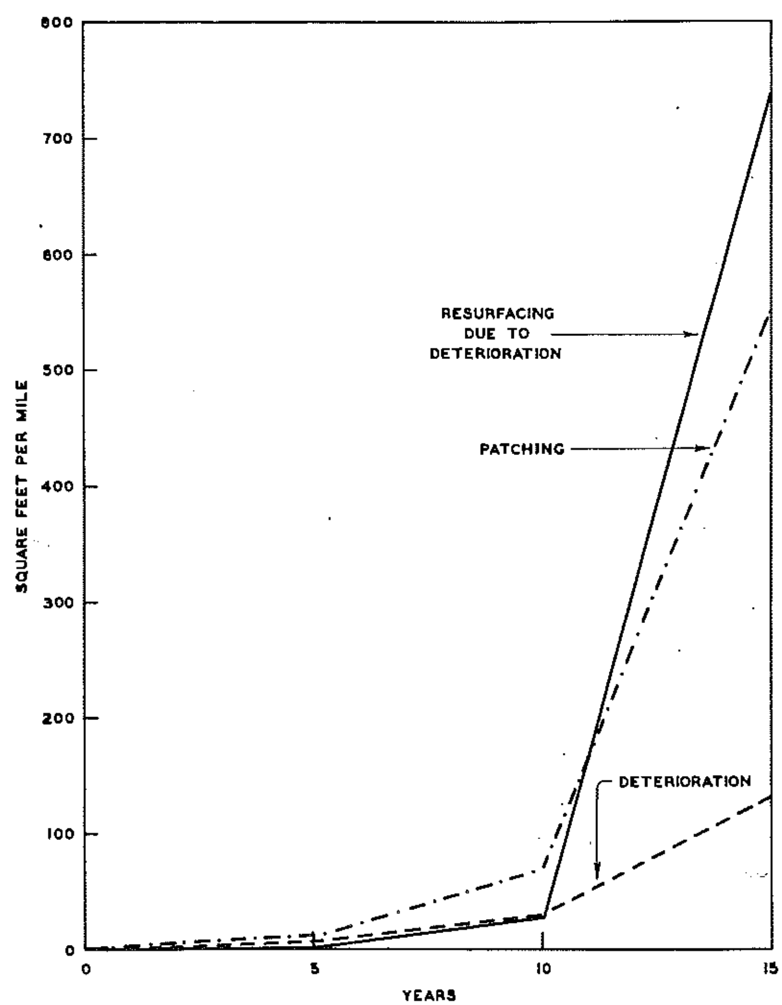


Figure 37. Resurfacing due to deterioration and patching based on 28 projects of two-lane pavement with service checks at 5, 10, and 15 years.

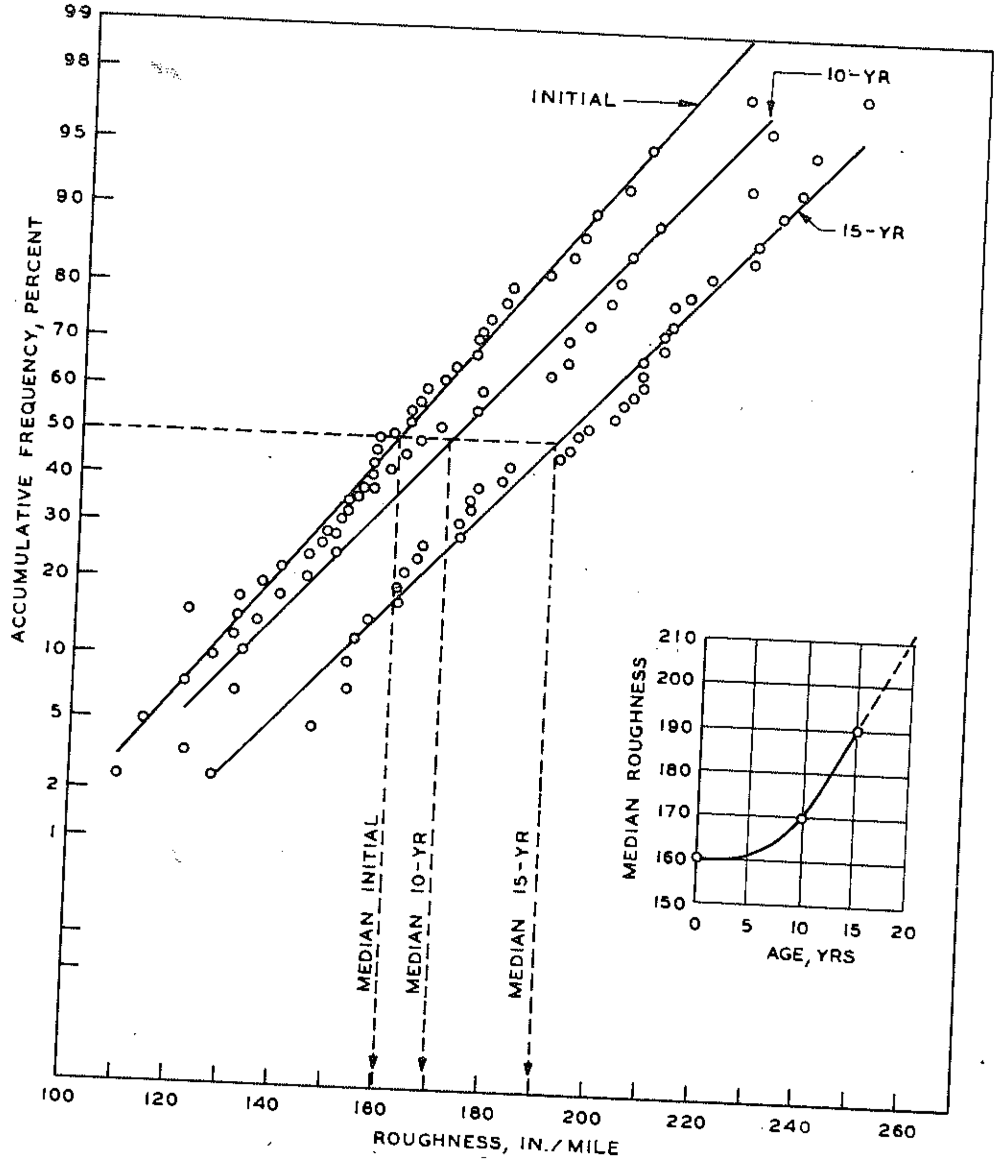


Figure 38. Progressive roughness increase with service.

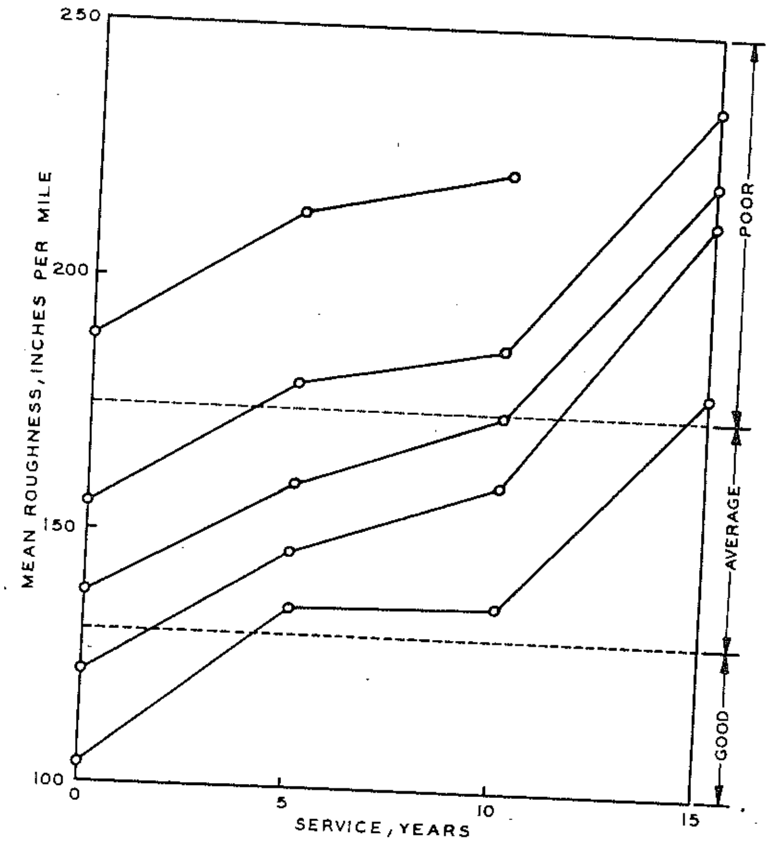


Figure 39. Progressive increase in roughness with service for five initial roughness categories.

Roughness

In addition to the specific survey variables described earlier, measurements of roughness were obtained for a number of projects when completed, and at one or more of the service periods of five, ten, or fifteen years. From one point of view, roughness may be considered as a summary of the information found in the other survey variables. A pronounced degree of deterioration as expressed in almost any of the condition survey variables could conceivably be reflected in roughness as measured by the Michigan roughometer. However, the usefulness of roughness as a general measure of performance is greatly diminished by the extreme variation of initial values. To be sure, if initial roughness were more uniform, this might possibly be used to measure structural deterioration. Michigan's experience, however, precludes this use, as shown in Figure 38. These cumulative frequency distributions show that initial roughness varies across almost the entire range of values obtained after ten and fifteen years of service. Many fifteen-year old pavements are smoother than many new pavements. The conclusion from Figure 38 is inescapable--a project's "roughness" depends more on its initial profile than its age or service. Only the median performance can be used to indicate roughness increase with service, where the median roughness for these projects with initial ten- and fifteen-year roughness surveys increased only 10 in. the first ten years of service and 20 in. the next five years. However, the overall variability of roughness values at each survey level is too large to permit the use of roughness as a research tool.

Considering roughness by itself, and bearing in mind its extreme variability, one can make some very crude comparisons. Figure 39 shows the progression of mean roughness for five arbitrary categories of initial values to the final fifteen-year determination. The general form of each curve suggests a plateau in the five- to ten-year range. Again it must be stressed that the values presented are averages and, consequently, represent general trends.

The differences in the rate of roughness increase may be attributed to the rate of increase of several condition survey performance variables. The more rapid increase in the first five years may be due to minor settlement which may or may not require mud-jacking. During the five- to ten-year period the roughness increase is less rapid but increases more rapidly in the ten- to fifteen-year period which correlates with the more rapid increase in pavement deterioration as reflected by a large increase in spalling, patching, blowups, and surface deterioration.

GENERAL PERFORMANCE PHILOSOPHY: THE NEED FOR A STRUCTURAL INDEX

It is often found that basic field or research variables are not directly and simply related to the property for which causal information is sought. The information contained in each basic variable may be totally lost because of the influence of other variables either not measured or controlled. Under these conditions, the use of a general index may be helpful. General indices are designed to amplify the desired information weakly expressed in the more basic and specific field variables. By a suitable weighting of these highly specific variables, it is hoped that an index can be constructed that will serve as a more powerful research tool in the investigation of more general properties. The structural performance of highways is an example of this problem: each condition survey variable measures a particular type of pavement distress, and therefore is not a complete measure of overall structural performance. In short, structural performance is a concept we cannot physically measure directly, but would like to define.

The manner in which condition survey variables are used in performance definition depends on the kind of performance considered important, as well as the methodological preferences of the investigator. Thus, there can be considerable disagreement concerning the proper formulation and use of these composite variables. Nevertheless, the simplicity and utility of a single, overall performance index justifies most any approach, provided its composition and assumptions are in clear view.

A pavement structural performance index can be used for the following purposes:

1. To summarize the essential information contained in the condition survey variables, thereby effecting a simplified measure of performance. While the index equation may be complex (i.e., some linear equation of the condition survey variables), once it is calculated, the structural performance of a length of highway is characterized by only one figure rather than a dozen or more.

2. To serve as a convenient tool in the search for possible "causes" of structural deterioration.

3. To anticipate the pavement's ability to continue in service.

4. To indicate the need for maintenance or improvements to forestall excessive deterioration.

Each condition survey variable is unique, and ideally could be uniquely associated with the relevant variables of design, materials, construction, or environment provided they were all known and measured with precision at the proper time. Because this amount and quality of information is usually not available, it is expedient to take the view that most condition survey variables express varying degrees of the same information, and this redundancy makes the complicated, tedious, and probably impossible consideration of each variable by itself unnecessary.

While indices can be computed for almost any length of pavement, computations for this study are for complete construction projects. Thus, the computed performance is considered general, in that it pertains to an entire project and not to a smaller subsection. This is not to say that short stretches of light or excessive deterioration associated with local conditions such as subbase, drainage, joint construction, etc., are of no interest. These factors, while affecting the general index, may not be linked with the overall conditions such as climate, materials, traffic, etc., set aside as possible determinents of performance. Consequently, these stretches are best investigated by the "case history" approach where all relevant local conditions are examined in depth. To this end, general performance indices conceivably may be used to spot extremes of deterioration, thereby reducing the number of projects requiring intensive investigation.

Performance indices used to measure highway structural deterioration[1] will display the same advantages and shortcomings encountered with their use in other areas of research. If one wishes to measure standard of living, economic activity, intelligence, or cardiac condition, he must decide on some abstract criterion of indirect measurement. What one usually finds available are many direct and specific measures, no one of which uniquely expresses the more general property subject to definition. Notwithstanding each measure's uniqueness, there is often good reason to relate these variables to the fundamental, more general property, with the presumed degree of relationship determining each variable's ultimate influence on the overall criterion.

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[1] Performance is generally considered to be a positive concept, and deterioration its polar opposite. The emphasis of this paper will be on the negative of performance, i.e., deterioration.

Subjective Rating Approach

The measurement of many highway properties should depend on subjective evaluations. These include aesthetic appeal, rideability, and even structural condition. In these cases a value judgment will be required that will not be identically made by all individuals. The AASHO Road Test, PSR (Present Serviceability Rating) is a case in point. Using a graphic rating scale of 1 to 5, professional and lay panels judged the general "serviceability" of a variety of pavements. By including persons of varying backgrounds on the lay panel, it was hoped that a stable evaluation approximating that of the general public could be produced by the averaging of individual responses. While an index so designed could be used for a variety of purposes (e.g., design or materials research) there are several disadvantages which limit its usefulness:[2] 1) no overt attempt was made to emphasize the relative seriousness of the various types of structural deterioration, and 2) it appears that both lay and professional panels are strongly influenced by longitudinal roughness; a highway property imperfectly correlated with structural condition.

In the Michigan study, it was considered worthwhile to design a rating index that would take into account the differential importance of the several survey variables. Admittedly, these differences are largely subjective--based on the professional judgment of an individual engineer. However, such an approach has the advantage of recording important conditions that may not enter into panel evaluations. Even if these conditions do not critically affect public acceptance at the time of rating, they may portend more serious deterioration and result in premature failure.

Subjective Rating Model

The form of this equation developed from a desired range of 0 to 10, with zero indicating a very poor pavement condition and 10 a perfect condition. All types of pavement deterioration obtained by condition surveys subtract from the perfect condition. Thus, it was a matter of subjective judgement to determine if a certain amount of longitudinal cracking was comparable in reducing the Performance Rating Factor (PRF) with another amount of deterioration, say, corner breaks or joint spalling. From the outset it was determined that the magnitude in the PRF reduction for one type of deterioration must be limited by an arbitrary maximum limit. The coefficients assigned to each term in the equation below do not directly represent weighting factors, influenced by the seriousness of the type of deterioration, but are a combination of this judgement factor and a scaling

[2] A complete list of problems with the PSI as seen from the psychological scaling view point can be found in reference (1).

factor to bring the various units of measurement (percent or average number) into a logical relationship.

The original intuitive equation (Model 1) was developed on the basis of pavement evaluation experience and with only a general review of the observed range in performance for each of the measured variables which collectively compose the rating. To study and adjust the coefficients and the maximum limits for each type of deterioration, 30 construction projects were used which had service records for five-, ten-, and fifteen-year periods. The distribution of the PRF values for these projects were studied as well as the changes in the distribution with service life.

$$PRF = 10-A\ [a_1\ (TC) + a_2\ (LC) + a_3\ (CB) + a_4\ (SP) + a_5 (BU) + a_6\ (PT) + a_7\ (MJ) + a_8\ (RS) + a_9\ (SC) + a_{10}\ (DT)]$$

where:

TC = average number of transverse cracks per slab

LC = percent of linear length of pavement with longitudinal cracking

CB = average number of corner breaks (exterior and interior) per mile of equivalent two-lane pavement

SP = average number of spalls per mile of equivalent two-lane pavement (This includes all spalls along transverse joints, longitudinal joints, cracks, or the interior of the slab.)

BU = percent of joints with blowups

PT = percent of pavement area with patches or structural replacement

MJ = percent of pavement area where mud-jacking was required

RS = percent of pavement area that had been resurfaced

SC = percent of pavement surface area with scaling

DT = percent of pavement surface area showing disintegration

MODEL 1			FINAL MODEL		
A =	1.0		A =	0.8	
a_1 =	1.0	TC term limited to max. of 4.0	a_1 =	1.00	TC term limited to max. of 5.0
a_2 =	0.25	LC term limited to max. of 1.0	a_2 =	0.060	LC term limited to max. of 1.5
a_3 =	0.25	CB term limited to max. of 0.5	a_3 =	0.04	CB term limited to max. of 1.0
a_4 =	0.03	SP term limited to max. of 2.5	a_4 =	0.008	SP term limited to max. of 2.0
a_5 =	0.015	BU term limited to max. of 1.0	a_5 =	0.08	BU term limited to max. of 2.0
a_6 =	0.60	PT term limited to max. of 2.5	a_6 =	2.5	PT term limited to max. of 2.5
a_7 =	0.30	MJ term limited to max. of 1.5	a_7 =	0.40	MJ term limited to max. of 2.0
a_8 =	0.20	RS term limited to max. of 4.0	a_8 =	0.10	RS term limited to max. of 10.0
a_9 =	0.10	SC term limited to max. of 1.0	a_9 =	0	SC term limited to max. of 0
a_{10} =	0.60	DT term limited to max. of 1.0	a_{10} =	4.00	DT term limited to max. of 2.0

Five models were tried, four of these were of the form shown above and the fifth used the log values of the terms representing deterioration in the equation. It should be noted that pavement scaling as a term in the rating was dropped in the final equation. This final model equation for the Performance Rating Factor was used as a single measure of project performance in various studies and analysis in later parts of this report.

Objective Rating Approach

In contrast with the AASHO Road Test's PSR (and therefore PSI) and PRF, an attempt will be made in this paper to devise a structural performance rating index on "objective" rather than "subjective" grounds (2). The technique of factor analysis (3) does not utilize either the differential subjective importance of the survey variables, nor public or professional subjective evaluation of serviceability. Rather, the empirical intercorrelations found among the survey variables provide the only basis for a performance equation. The key methodological assumption is that if the survey variables linearly measure in varying degrees general structural performance, they will be intercorrelated accordingly. The degree to which a given variable is correlated with the others in a group reflects the extent to which it expresses the "common" performance characteristic. Because this method of performance fabrication does not utilize any form of subjective evaluation its chief contribution is in delineating basic, non-judgmental categories of pavement deterioration which simplify the search for assignable causes. For the present study, the survey variables are "reduced" first to a single, more general category of performance (deterioration) from which is developed a single performance index. While more specific categories can be subsequently extracted, their generality will decrease and they will account for diminishing amounts of survey variable intercorrelation.

By stressing the subjective aspect of performance, the PSI and PRF approaches could weight the survey variables disproportionately to the effects of the underlying "causes" of structural distress. While the same argument could be advanced against an index derived from factor analysis, one should remember that statistical association provides the investigator with a necessary, though not sufficient, condition for the assumption of "casual" relationship. The association assumed by the factor analysis approach is simple linear correlation.

Another advantage of "objective" rating is that the sets of intercorrelations for the five-, ten-, and fifteen-year survey periods can be considered separately. Consequently, the changing pattern of intercorrelations reflecting the evolutionary or retrogressive importance of each variable in respect to the underlying causes of deterioration can be acknowledge in different rating equations for each survey period. This is not usually possible with any of the subjective approaches. Both the PRF and PSI equations are invariant with time, thereby disallowing corrections for changes in each variable's relationship to general performance or to the underlying system of causes. On the other hand, there are occasions when strict continuity of performance is desirable, requiring a uniform performance measure. PRF and PSI can be computed at any time in a project's life and, taken sequentially, can provide a historical record of performance. Consequently, projects can be compared on the basis of their performance time histories. This latter advantage was considered important, and the present analysis was conducted accordingly using a single performance equation for all survey periods. The various types of indices are compared in Table 1 and Figure 40.

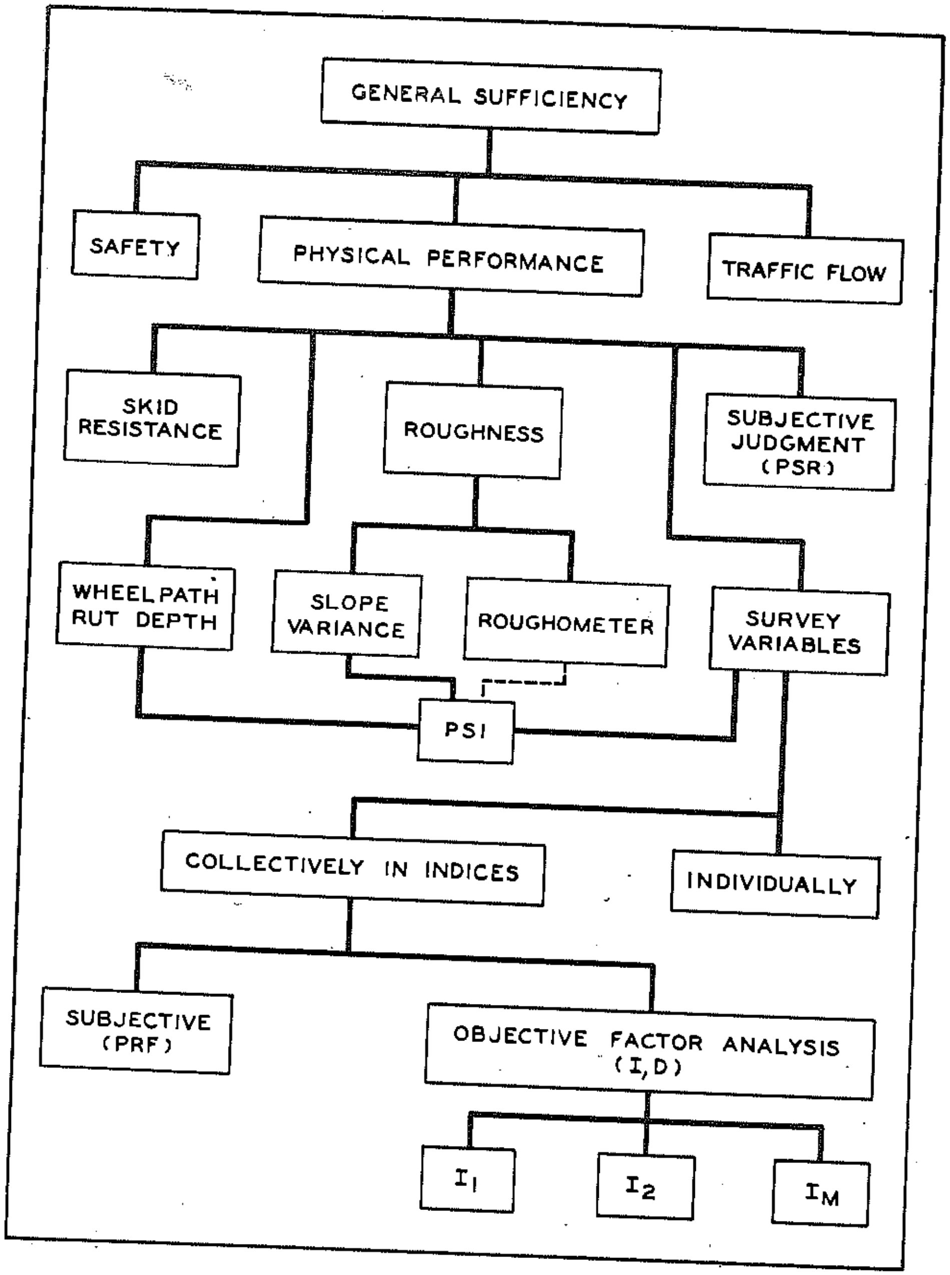

Figure 40. Alternative methods of rating pavement performance.

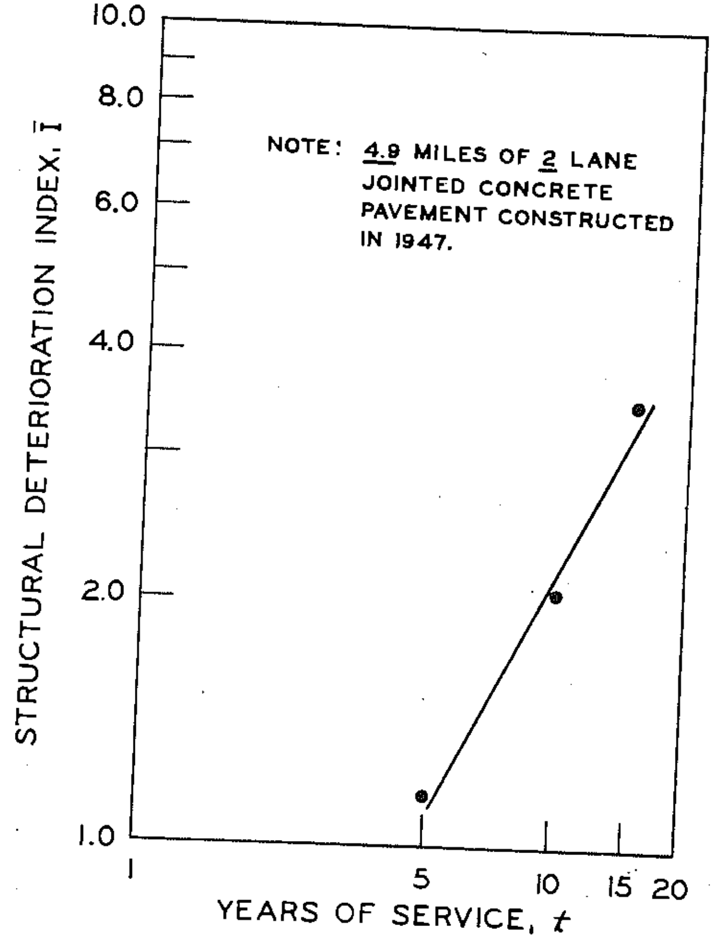

Figure 41. Time index history for construction project 17-7-66.

Objective Rating Model

The principal axis method of factor analysis based on the survey variable intercorrelation matrix yielded the following structural deterioration index (I):

$$I = [.004\ (LC) + .005\ (TC) + .006\ (D) + .106\ (BU) + .011(CBJ)$$

$$+ .410\ (CBTC) + .013\ (SJ) + .359\ (STC) + .043\ (SE)] \ldots\ldots\ (1)$$

where:

LC	=	longitudinal cracks per mile
TC	=	transverse cracks per mile
D	=	disintegration in square feet per mile
BU	=	blowups per mile
CBJ	=	corner breaks at transverse joints per mile
CBTC	=	corner breaks at transverse cracks per mile
SJ	=	spalls at transverse joints per mile
STC	=	spalls at transverse cracks per mile
RS	=	remaining spalls per mile

While a pavement in perfect structural condition would have an index value of zero, there is no limit to the degree of deterioration measurable with the scale. Unlike the PSI, the structural index has ratio scale status, and therefore can be mathematically manipulated without attention to scaling assumptions.

Index values can be computed for any year for which a condition survey is available. Substitution of a series of surveys taken during a project's service life, in equation (1), will show the structural deterioration trend as well as the project's relative condition at any service period. Figure 41 shows the structural deterioration index history for a project for which the five-, ten-, and fifteen-year surveys are available. It is characteristic of this project, and others in general, that the index history plot is nearly linear when plotted on log-log coordinates. Therefore, it is assumed that a close approximation to the index history could be made with the following power function:

$$\log \hat{I} = A \log t + \log B \quad \text{or}$$

$$\hat{I} = Bt^{A} \qquad (2)$$

where:

$\hat{I}$ = structural deterioration index estimate

t = service time in years

A, B = fitting constants unique to each project, and determined by least squares

TABLE 1
PROPERTIES OF VARIOUS PERFORMANCE INDICES

Property	Present Serviceability Index (PSI)	Performance Rating Factor (PRF)	Structural Deterioration Index * (I)	Structural Depreciation Index (D)
Based on Engineering Judgement as to Seriousness of Deterioration	No	Yes	No	No
Based on Public Evaluation of Serviceability	Yes	No	No	No
Allows Rating of Progressive Deterioration	Yes	Yes	Yes	Yes
Coefficient Weights Proportional to Intercorrelation	----	No	Yes	Yes
Allows for Separate Indices for Different Types of Deterioration	No	No	Yes	Yes
Unique-Not Dependent on Investigator	Yes	No	Yes/No*	Yes/No*

* The approach is unique if each investigator uses the same mathematical techniques. However, there is a variety of approaches with variety of solutions.

The structural deterioration index I, and its estimate $\hat{I}$, can be computed by equations (1) and (2) for any specific time, t. However, condition at a point in time is generally of minor interest; one usually wishes information on the rate of deterioration so that performance can be evaluated. In the present case, the negative of performance will be called "depreciation." To evaluate total depreciation, D, over each project's service life, deterioration was summed over time as follows:

$$d\hat{D} = \hat{I}\,dt = Bt^{A}\,dt \qquad \text{and}$$

$$\hat{D}_T = B\int_o^T t^A\,dt = \frac{B}{A+1}\,T^{A+1} \qquad \text{(3)}$$

where:

$\hat{D}_T$ = estimate of structural depreciation from time of construction to terminal rating

T = elapsed time in years to terminal rating

Using equation (3) one can evaluate each pavement's structural depreciation from construction to any time up to fifteen years. [3]

The structural depreciation index, D, accomplishes first the pooling of correlated information (survey variables) on deterioration into a time-dependent measure of pavement condition, and second, the summarization of deterioration over service life. Thus, the deterioration index I, and the depreciation index D, provide single measures of negative structural condition and performance, respectively. It was hoped that these very general measures would facilitate the search for associated materials, environment, and construction variables.

Certain fundamental properties of the three performance indices previously discussed are compared in Table 1. In the development and use of performance rating indices, and before attempting to select one of these indices to compare with material and construction variables in a cause and effect relationship, it is of interest to determine if these different factors correlate. Thirty projects with fifteen years of service and 55 projects

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[3] Survey data beyond fifteen years were not available; therefore, equation (2) can be applied only to this period.

with ten years of service were compared on the basis of PRF and PSI values. The values of PRF varied from 1.75 to 9.1 for fifteen-year projects and from 4.8 to 9.7 for ten-year projects. Corresponding values of PSI were from 2.1 to 3.7 and 2.0 to 3.8, respectively. Although the graphs are not shown, the resulting scatter of points for each service period was so broad that it was apparent no correlation of these two rating methods was possible. This is not unexpected for, as discussed previously, these rating systems are measuring different aspects of performance: the one physical indications of pavement deterioration and the other, primarily, roughness.

OBJECTIVE III

TO DETERMINE THE EFFECT OF VARIOUS SOURCES OF MATERIALS USED IN THE PROJECTS ON THE PERFORMANCE CHARACTERISTICS OF THE PAVEMENT

ANALYSIS OF CONSTRUCTION, MATERIALS, AND ENVIRONMENTAL VARIABLES

Once construction, materials, and environmental variables have been classified into levels or categories, one can examine the differences in performance (as measured by I, D, PRF, or PSI). In the present study, PRF and I were used in a statistical test (analysis of variance) designed to evaluate performance differences in terms of the random fluctuations of measurement always present in field examinations of this type. The variables listed below were examined as potential causes of pavement deterioration.

Average Yearly Rainfall

The geography of the State was divided into regions of high yearly rainfall (above the median) on the basis of all available weather station data. Projects were classified according to their geographic region to allow comparative examination of their performance.

Average Daily Temperature Differential

A method of classification similar to that for average yearly rainfall was applied to separate geographic regions of relatively high and low average daily temperature changes.

Contractor

In an attempt to determine if the contractor's workmanship had a significant effect on subsequent pavement performance, the Construction Division was requested to rate 34 paving contractors performing work on projects under study into three categories, "excellent," "good," and "fair." It was suggested that this rating should be based on workmanship during the period of 1947 through 1954 when these projects were under construction. The Construction Division rated 12 contractors in the "excellent," 10 in the "good," and 12 in the "fair" categories.

Subgrade

Each project was located on a state trunkline map over which was superimposed a map indicating regions of good, fair, or poor subgrade. Each project was assigned a value of 1, 2, or 3 according to the category shown on the map (Fig. 42). If a project did not lie exclusively within any one of the categories it was handled in either of two ways: 1) if 90 percent of the project was in one category location, it was given that category, or 2) if less than 90 percent was located in any one category, all pertinent grades were recorded.

Construction Period

Median construction dates (the date halfway between the earliest and latest concrete pour dates for each project) were divided into two categories--those falling in the summer months of June, July, and August, and those falling in the remaining months prior to, or subsequent to, the summer months.

Average Daily Commercial Traffic

The median of all commercial average daily traffic volumes was used to delineate "high" and "low" traffic categories. Traffic survey data were available only for 1947, 1955, 1957, and 1961, the years used to produce the overall average value for each period.

Coarse Aggregate

A convenient aggregate classification in the present study is made available through carbonate content tests. Carbonate content tests were not available from the aggregate incorporated in the construction projects. However, in subsequent years after construction, coarse aggregate from various pits was sampled and carbonate content tests were conducted. Even though these tests were pit samples taken some years after the pit aggregate was used in construction, it was felt that the carbonate content was reasonably representative of that used in the construction project. Aggregate categories based on estimated carbonate content provide the basis for classification and subsequent performance evaluation. These categories were 80-100 percent carbonate and 0-60 percent carbonate.

Figure 42. Distribution of subgrade types, Lower Michigan.

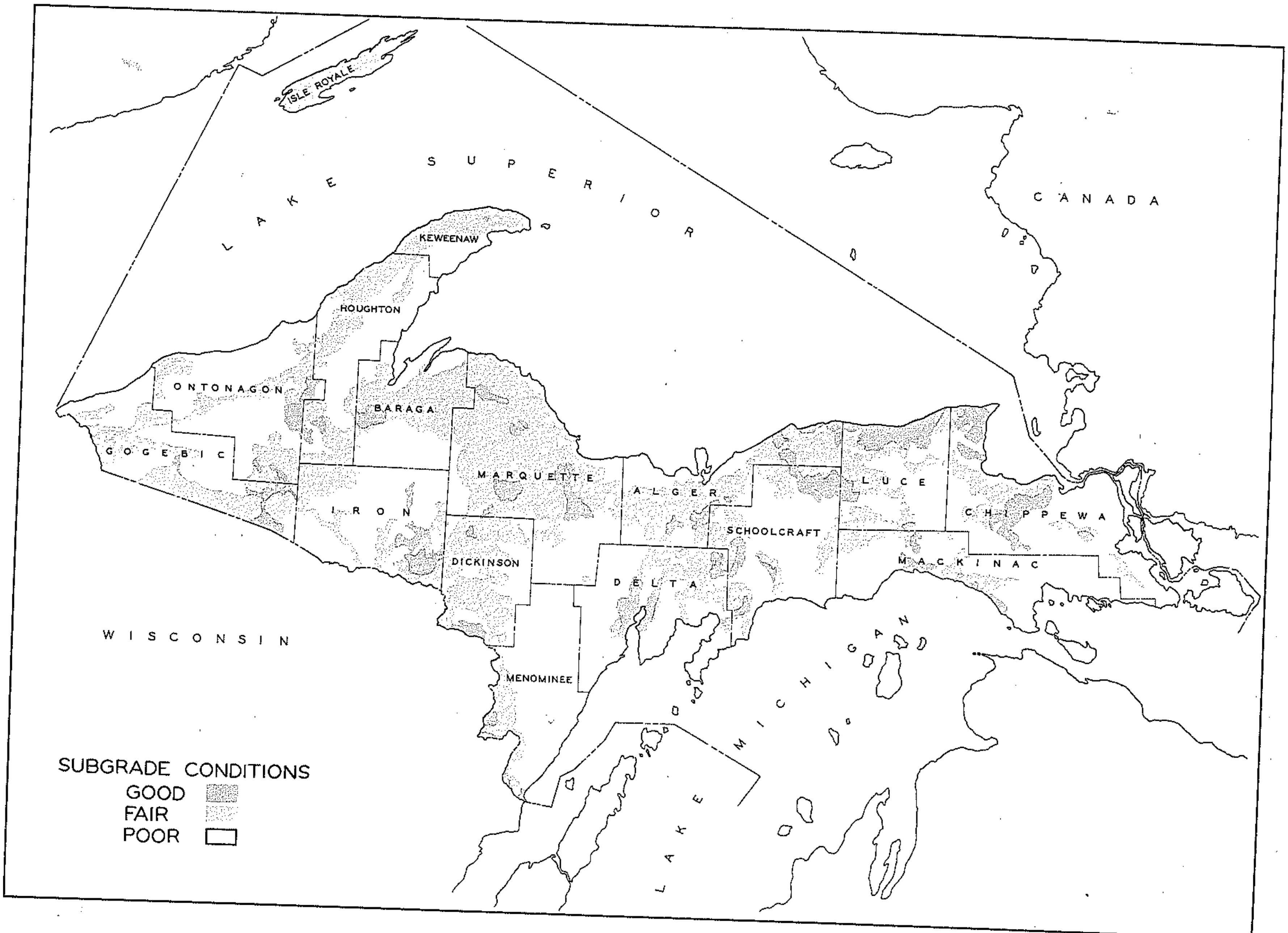

Figure 42 (cont.). Distribution of subgrade types, Upper Michigan.

DISCUSSION OF RESULTS WITH GENERAL PERFORMANCE INDICES

A general analysis of variance suggested the existence of relationships between average daily commercial traffic (ADCT), coarse aggregate, and deterioration, I. As shown in Figure 43, gravels containing high proportions of carbonates and aggregates composed of pure crushed limestone or dolomite perform best (generally lower I values) over the fifteen-year service period. At fifteen years of service, the average I is about 2-1/2 times greater for those projects built with gravels of relatively low carbonate content. More refined examination of these aggregate groups shows that further subdivision is possible: of the projects constructed with aggregates containing 80-100 percent carbonates, those using 100 percent pure crushed limestones and dolomites which were studied here had the smallest I values. Also, the pure gravels containing no carbonates performed a bit better than other aggregates in the 0-60 percent carbonate group. This suggests that aggregate heterogeneity and not merely carbonate contents more closely associates with pavement performance.

A rough attempt to quantify this possibility was made using the following formula:

$$H = \sin (P\pi)$$

Where H is defined as heterogeneity, and P is the proportion of carbonate in the coarse aggregate. Thus, 100-percent pure crushed limestone and 100-percent pure igneous rock gravel will have an H value of 0.0, while an aggregate composed of 50 percent carbonate and 50 percent other rock types will have the maximum H value of 1.0.

Figure 44 shows that, in general, performance tends to deteriorate as carbonate-gravel heterogeneity increases. This is especially the case after fifteen years of service where aggregate classes show wide performance variance. However, very general graphic comparisons, such as those just discussed, are rarely sufficient in themselves to pinpoint the causal mechanism involved. Clearly, many local conditions such as faulty joint construction, subgrade support, etc., relate to each particular case and type of deterioration and one cannot generally speak of one cause alone.

Moreover, even if a single factor is identified, it may not be causally important, but only statistically associated with the real cause of poor performance. Because the relationship is empirical, the possibility always exists that the statistical finding is misleading; the real mechanism being masked by the complex interrelationship between statistically associated variables. The relationship of aggregates to performance is a case in point: the poorer performance of the heterogeneous aggregates could be due to the differential thermal expansion rates of the several components in the aggregate, or to some other variable associated with aggregate heterogeneity. Past research indicates that the latter possibility is more likely (4). Cherts, soft, non-durable particles, hard absorbent particles, etc., have long been suspected as causes of various kinds of pavement deterioration. The absorption and expansion properties of the materials

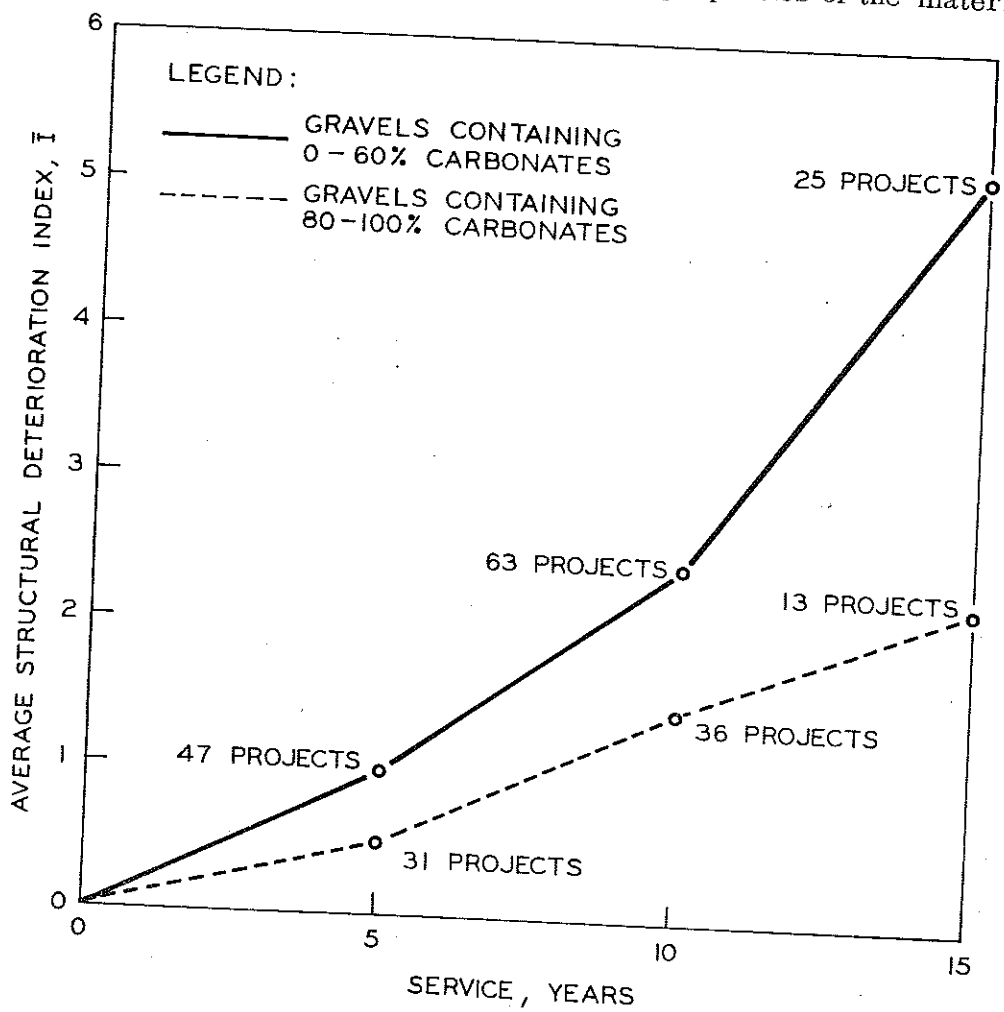

Figure 43. General performance as affected by two aggregate classes.

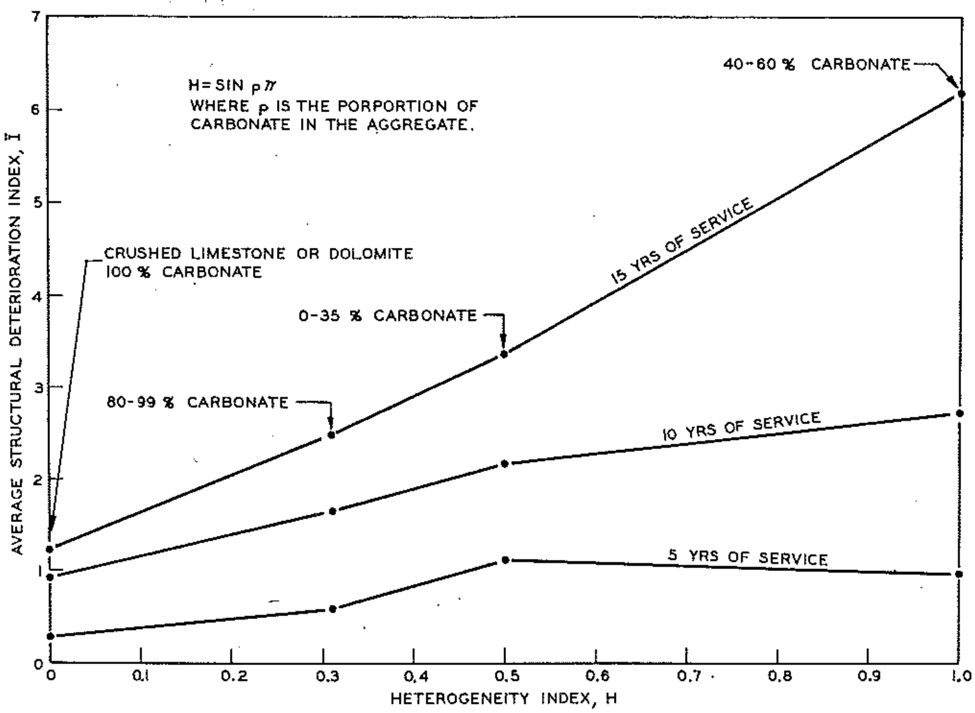

Figure 44. The effect of heterogeneity index on the structural deterioration index.

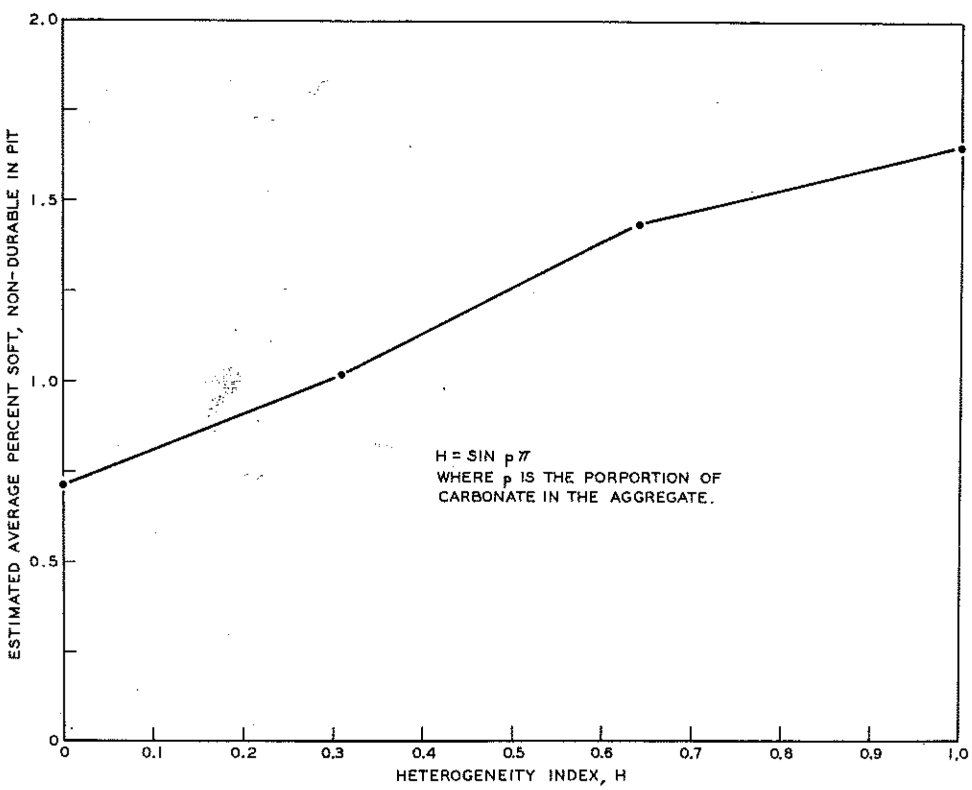

Figure 45. Relationship between coarse aggregate heterogeneity and soft, non-durable content.

have inspired attempts at performance forecasts through laboratory freeze-thaw tests. Soft, non-durables[4] seem to be present in Michigan's heterogeneous aggregates (Fig. 45) and may, as suggested by other research, be responsible for the structural performance differences encountered (5, 6, 7, 8).

These findings are similar to those of a blowup study to be discussed later which also shows a relationship between aggregate type and performance. Other investigators have also found associations between aggregates and various types of deterioration (9, 10, 11, 12). While the gravels and limestones could perform differently, past research indicates that the problem probably lies with the so-called deleterious particles found to various degrees in these aggregates.

Examination of Soft Particle and Chert Data

Because of past work in these areas, and the evidence shown in Figure 45, it was considered unlikely that the simple gravel-limestone aggregate classification used in the preceeding examinations would sufficiently define the performance-materials relationship. For this reason, PRF, I, and PSI values were examined in connection with available soft particle and chert information.

Projects for which field tests were available provided the basis for analysis. From these records, averages of test results for soft, non-durable and chert percentages were obtained and used as an estimate of the overall content of these materials in the coarse aggregate. Neither PRF, I, nor PSI showed significant dependence on the chert content; however, correlations appeared to exist between soft, non-durable content and PRF and I. Table 2 shows the results of a PRF analysis of variance on divided expressway for which soft, non-durable content and commercial traffic volume data were available. At the five-year service level, neither soft, non-durable content, traffic, roadway, or lane show PRF differences large enough to be considered significant.

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4 As defined by Michigan specifications, soft particles include shale, soft sandstone, ochre, iron-bearing clay, weathered schist, shells, floaters, partially disintegrated particles, cemented gravel and any other particles which are structurally weak or which fail to meet the soundness test. Michigan Specification for 4A and 10A aggregates used in concrete pavement: 3 percent maximum.

The ten- and fifteen-year PRF indices show noteworthy differences presumably due to traffic, lane, and soft content (see Table B-1, Appendix 2). Table 2 presents a summary of the percentage variance contributions attributable to the analysis variables and their interactions. Of special note is the sharp decrease in unexplained variance between contracts and lanes after the five-year survey period. The effects of soft, non-durables in the aggregate together with the overall and between-lane traffic patterns apparently become strong enough after five-years of service to account for much of the between roadway, contract, and lane variance. It must be remembered that because of classification restrictions, only four contracts (three service periods, divided roadways, and all four combinations of soft, non-durables and traffic values) have been herein examined. Consequently, these results are indications only; further analysis being necessary to establish confidence.

TABLE 2

COMPONENTS OF VARIANCE FOR PRF - (Divided Expressways)*

Years of Service	Percent Contribution to Total Performance Variance									
	Explained Variance							Unexplained Variance		
	Soft Non-durable	Traffic Volume	Soft-Traffic Interaction	Lane	Lane-Soft Interaction	Lane-Traffic Interaction	Lane-Soft-Traffic Interaction	Between Roadway	Between Contract	Between Lane
5	0	0	12	0	2	0	0	0	71	15
10	10	47	0	14	0	11	0	1	15	2
15	24	7	11	14	3	7	2	15	16	1

* Table B-1 (Appendix B) shows the full analysis of variance for the PRF. Table B-2 (Appendix B) shows the same analysis applied to 5-, 10-, and 15-year PSI data.

In general, (except for soft, non-durable at the fifteen-year level) the PSI index does not appear sensitive to the traffic or aggregate variables. There are slight differences, but again they are not large enough to be considered significant. It is known that the PSI depends largely on roughness (13), a performance measure only tenuously related to structural distress. This is because roughness is not particularly sensitive to such structural deterioration variables as transverse cracking because reinforcing has effectively prevented faulting. Also, other valuables such as corner and centerline spalling are generally not picked up by the roughometer wheel.

For graphical purposes, all projects were classified as to high or low soft, non-durable contents and high or low average daily commercial traffic (ADCT). The class divisions were based on the soft, non-durable and ADCT medians. Figures 46 through 51 show averages for five-, ten-, and fifteen-year periods of PRF, I, and PSI for the above classifications using

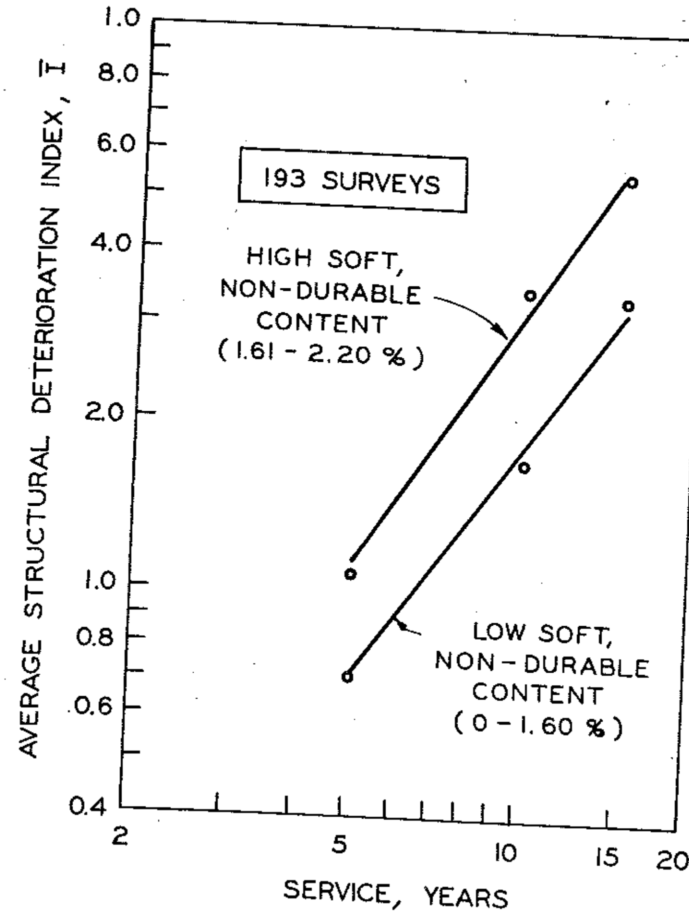

Figure 46. Average structural deterioration for projects with low and high soft, non-durable content.

data from all available non-divided roadway projects. Notice that low ADCT and low soft, non-durable values are associated with better performance for both indices. However, PRF and I are more affected than PSI. To show the effects in more detail, moving averages of I are plotted against both ADCT and soft, non-durable for the five-, ten-, and fifteen-year service periods in Figures 52 and 53. Log $\hat{I}$ appears to increase with both log soft, non-durables and log ADCT at about the same rate with each service period. These moving averages, while showing predominent trends, remove considerable scatter from the data--correlations are of the order of only 0.40 to 0.50. Correlations would be higher but for only five or six projects (out of nearly 100). These projects either had low traffic or low soft content (or both), and showed excessive deterioration early in service life. No reason could be found, and it is assumed that local soil conditions are responsible.

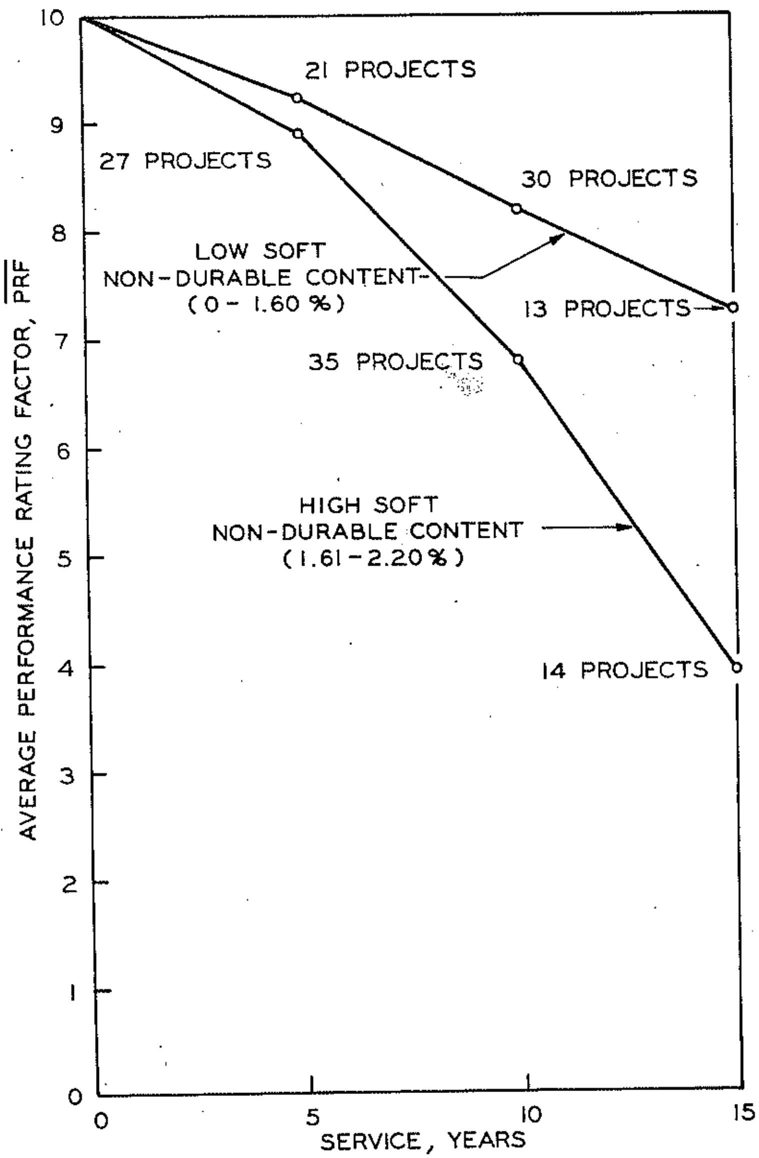

Figure 47. Performance rating factor for projects with low and high soft, non-durable content.

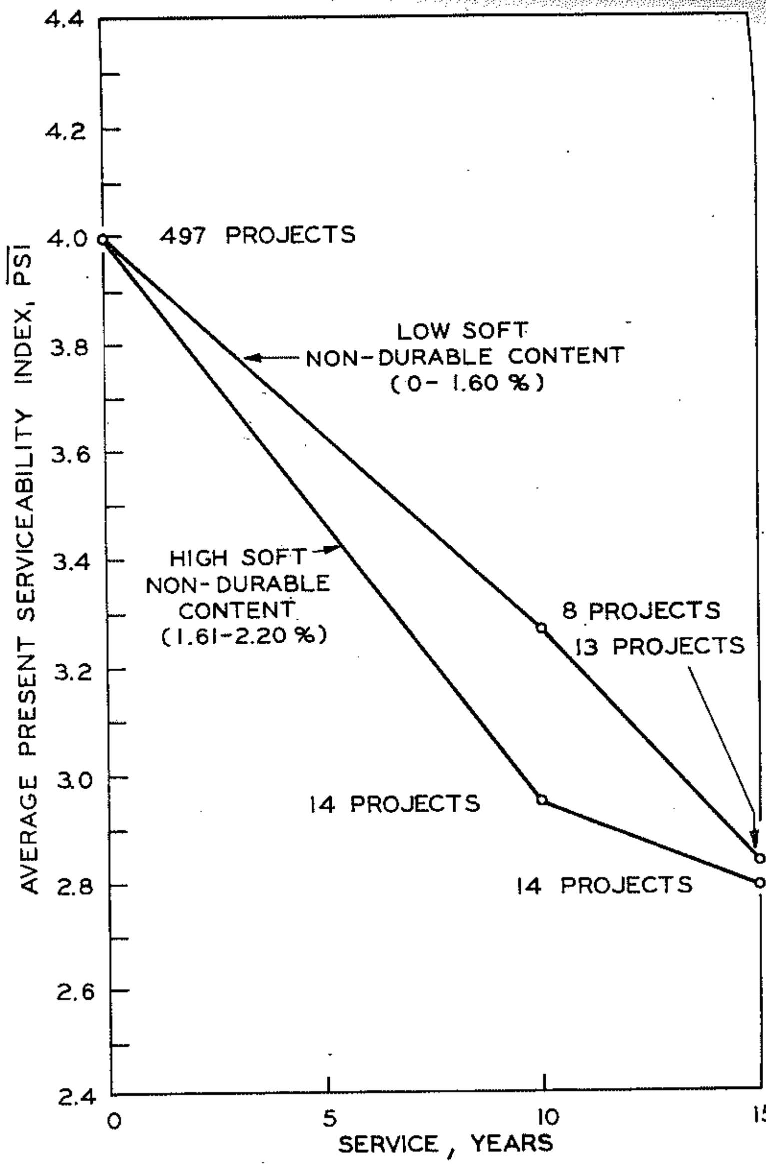

Figure 48. Present serviceability index for projects with low and high soft, non-durable content.

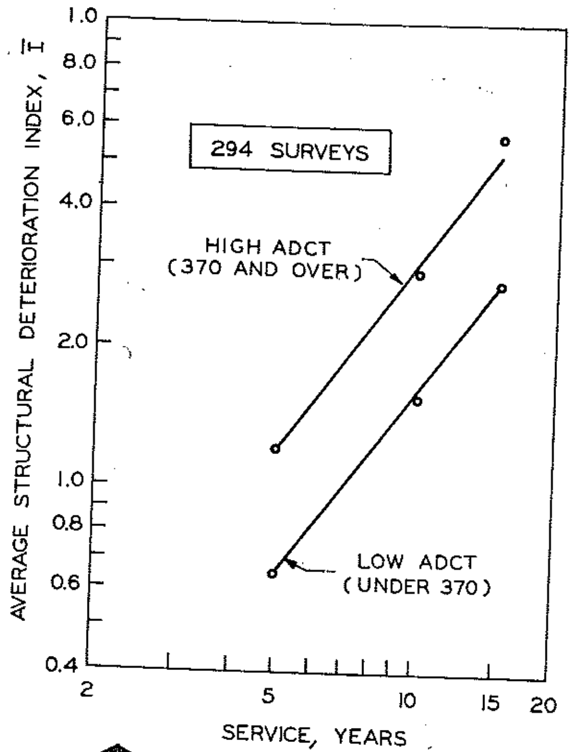

Figure 49. Relationship between average daily commercial traffic volume (ADCT) and average structural deterioration (I).

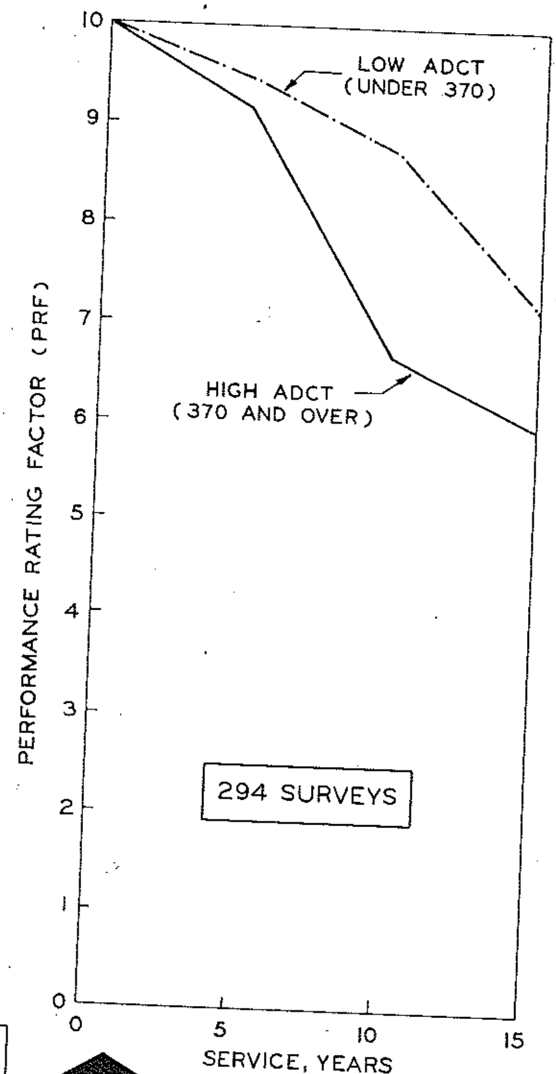

Figure 50. Relationship between average daily commercial traffic volume (ADCT) and average performance rating factor (PRF).

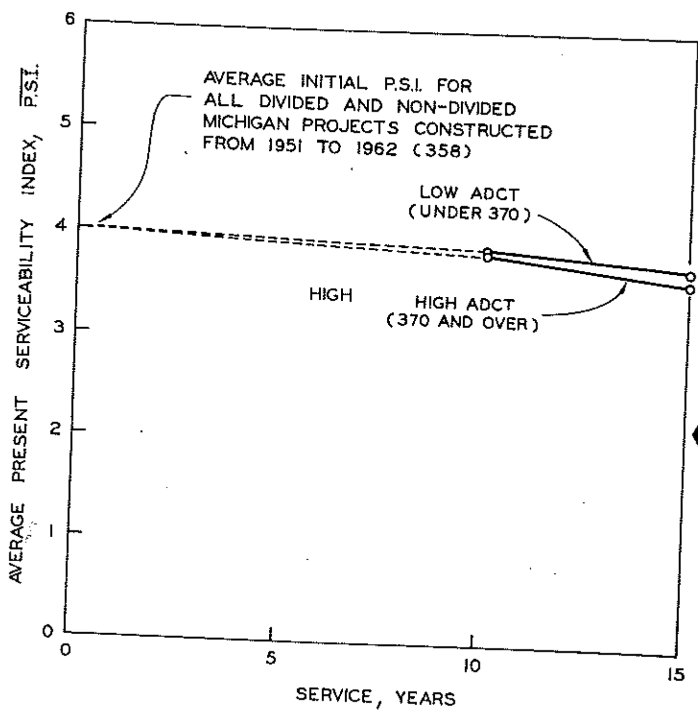

Figure 51. Relationship between average daily commercial traffic volume (ADCT) and average present serviceability index (PSI).

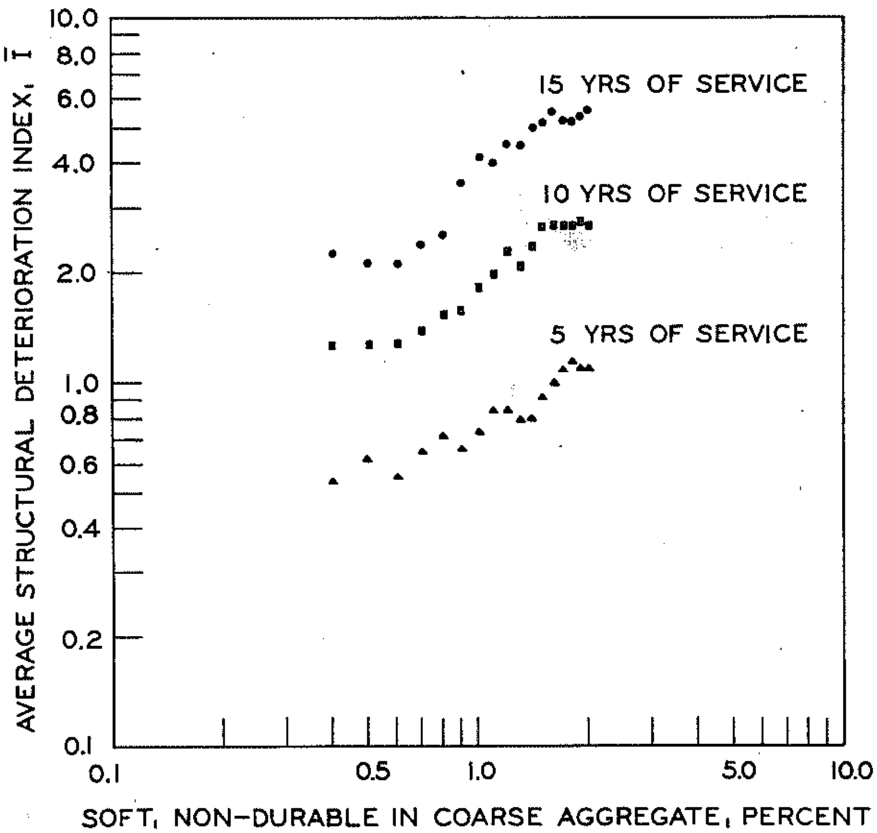

Figure 52. Relation between soft, non-durable and average structural deterioration.

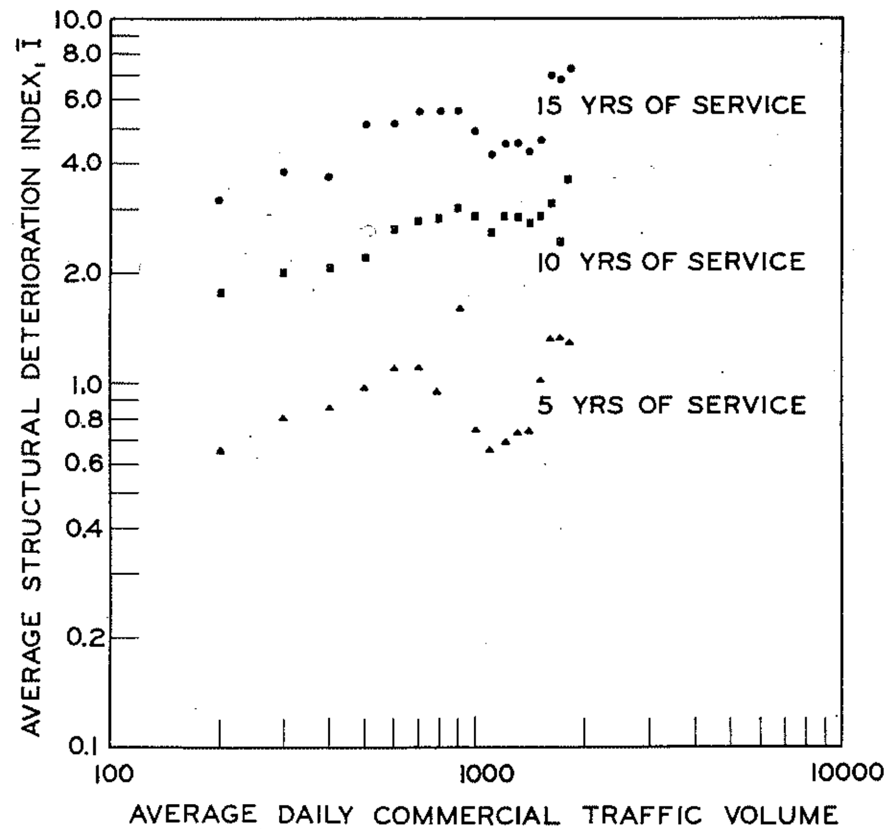

Figure 53. Relation between average daily commercial traffic volume and average structural deterioration (I).

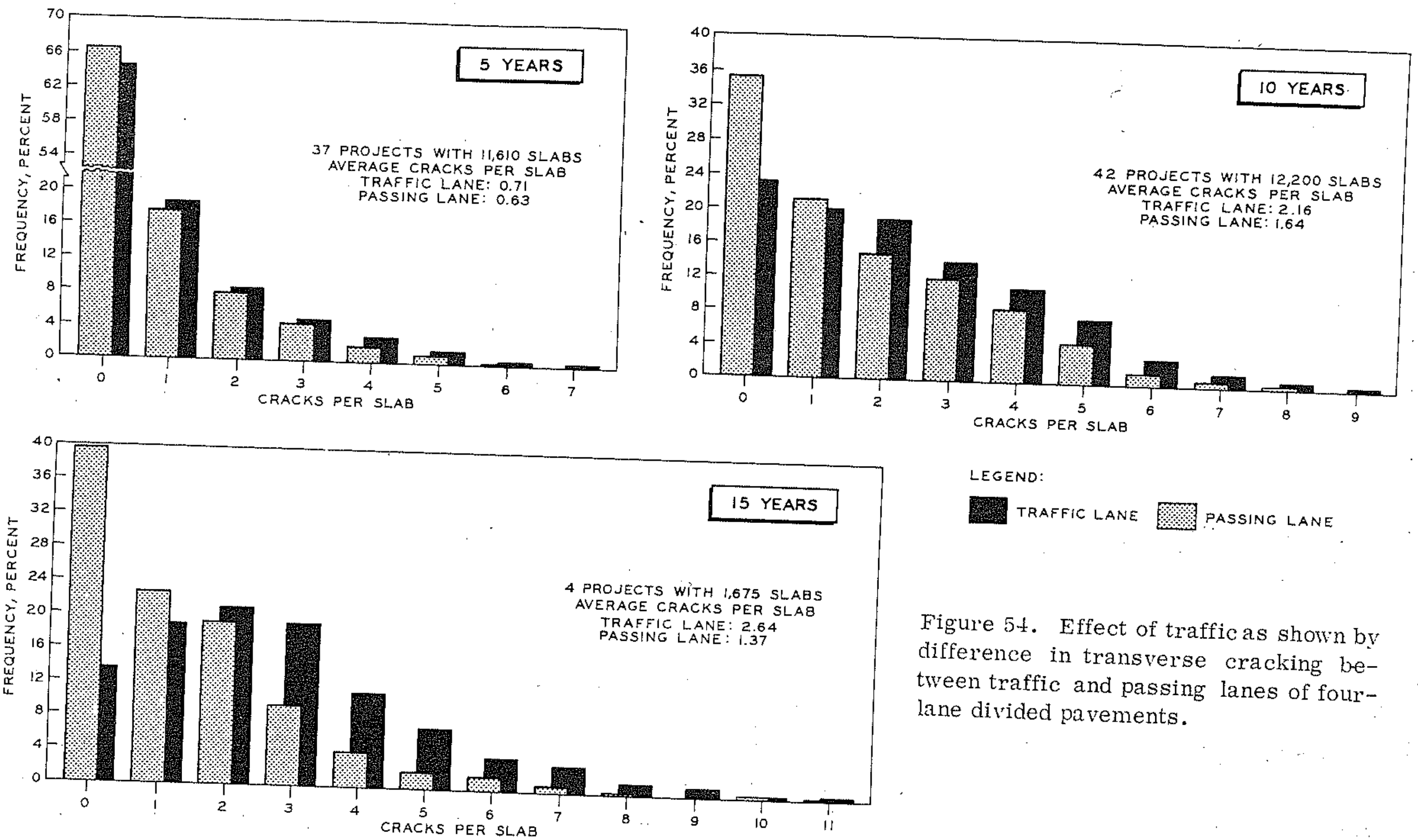

Figure 54. Effect of traffic as shown by difference in transverse cracking between traffic and passing lanes of four-lane divided pavements.

The data used for commercial traffic resulted from a single 24-hour sample taken during the service life of each project. Because these data are so weak, the influence of traffic on performance requires further examination. Four-lane divided expressways provide an excellent opportunity to evaluate lane performance differences attributable to differential usage. For these pavements, all construction, environment, design and materials variables are identical thus the difference in performance between traffic and passing lanes can be unequivocally assigned to differences in traffic load. A quantitative between-lane difference in traffic load was generally not available for the projects examined. However, it is well known that traffic lanes experience more loading than the corresponding passing lanes. In terms of equivalent 18 kip axle loads, the traffic lane can generally be assigned from 70 to 90 percent of the total in one direction.

The difference in transverse cracking between traffic and passing lanes of four-lane divided pavements is shown in Figure 54. It should be noted that the effect of traffic on performance of traffic lanes in comparison with passing lanes is small for five years of service; but with increased service, at ten and fifteen years, the difference is much more apparent. As a result of this study, all available construction projects on divided highways--including those which were surveyed after the initial cut-off data for this study--were examined to obtain a broader statistical basis for determining the effect of traffic.

In Figure 55 the average incidence of transverse cracking is shown to be higher for the traffic lane at five-, ten-, and fifteen-year service periods for all construction projects built as divided expressways. The traffic lane has, on the average, 62 percent more transverse cracking after fifteen years service than does the passing lane. The difference is even greater, as shown in Figure 56, for average longitudinal cracking; after fifteen years service the traffic lane has nearly twice as much cracking as the passing lane, 59 lin ft for 100 slabs as compared to 30 lin ft for the passing lane. This same difference in performance after fifteen years of service between traffic and passing lane can be noted for external corner spalls in Figure 57 (36 percent increase in traffic lane); internal corner spalls in Figure 58 (29 percent increase in traffic lane); amount of patching in Figure 59 (20 percent increase in traffic lane); and amount of pavement deterioration in Figure 60 (61 percent increase in traffic lane). It should be noted that in every case the traffic lane performed more poorly as a result of greater traffic than the passing lane.

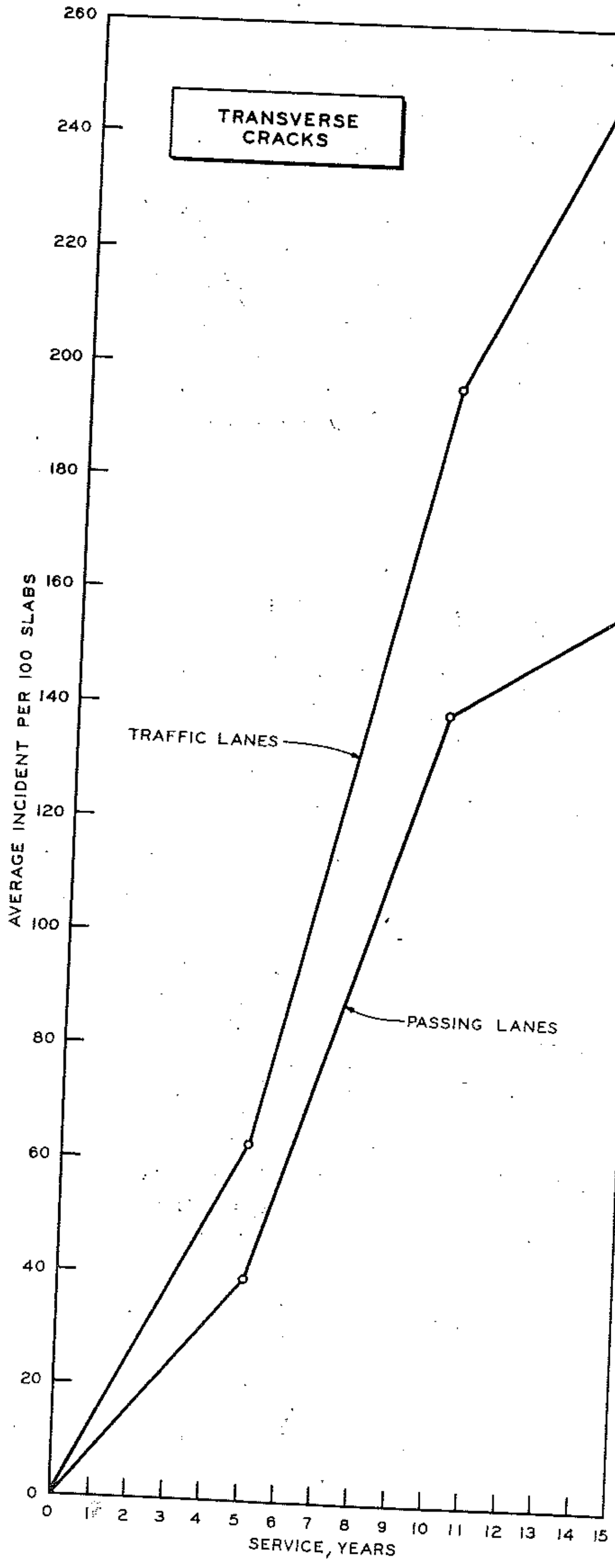

Figure 55. Effect of traffic on transverse cracking.

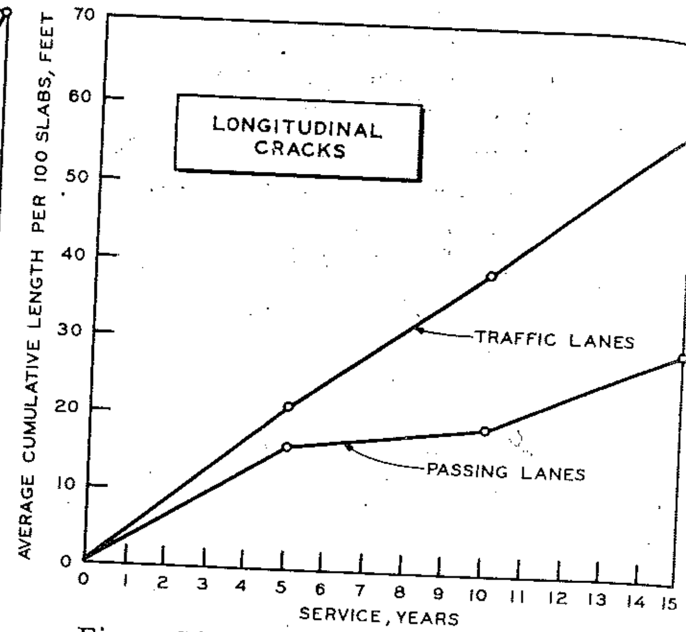

Figure 56. Effect of traffic on longitudinal cracking.

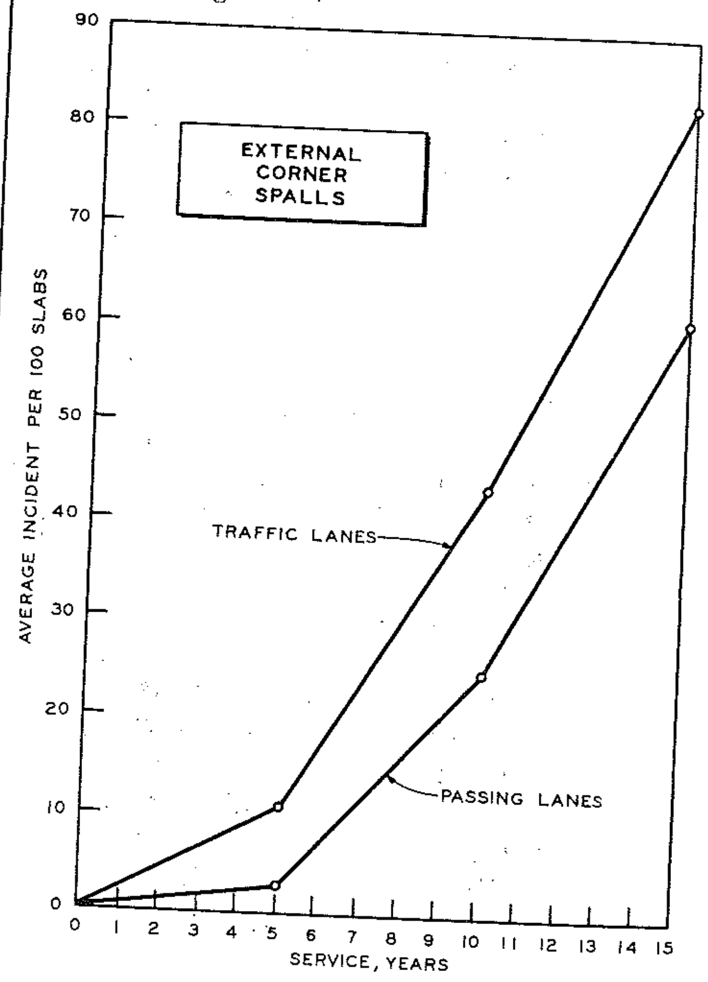

Figure 57. Effect of traffic on external corner spalls at transverse joints.

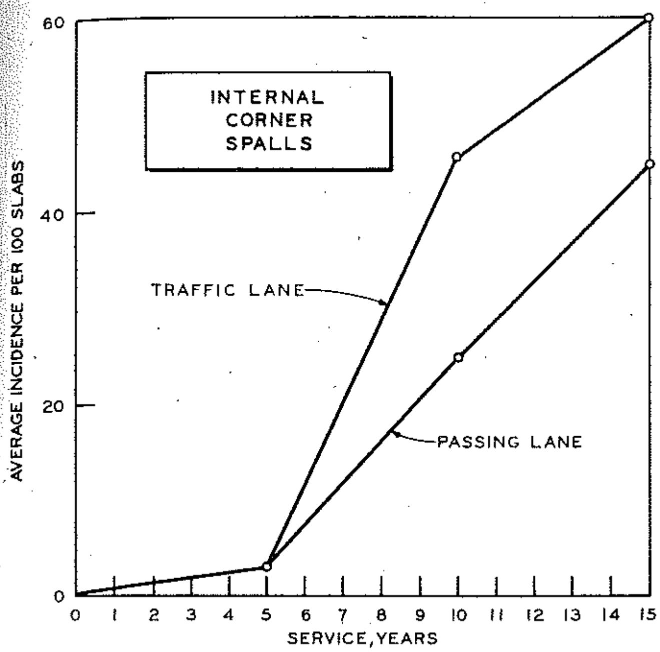

Figure 58. Effect of traffic on internal corner spalls at transverse joints.

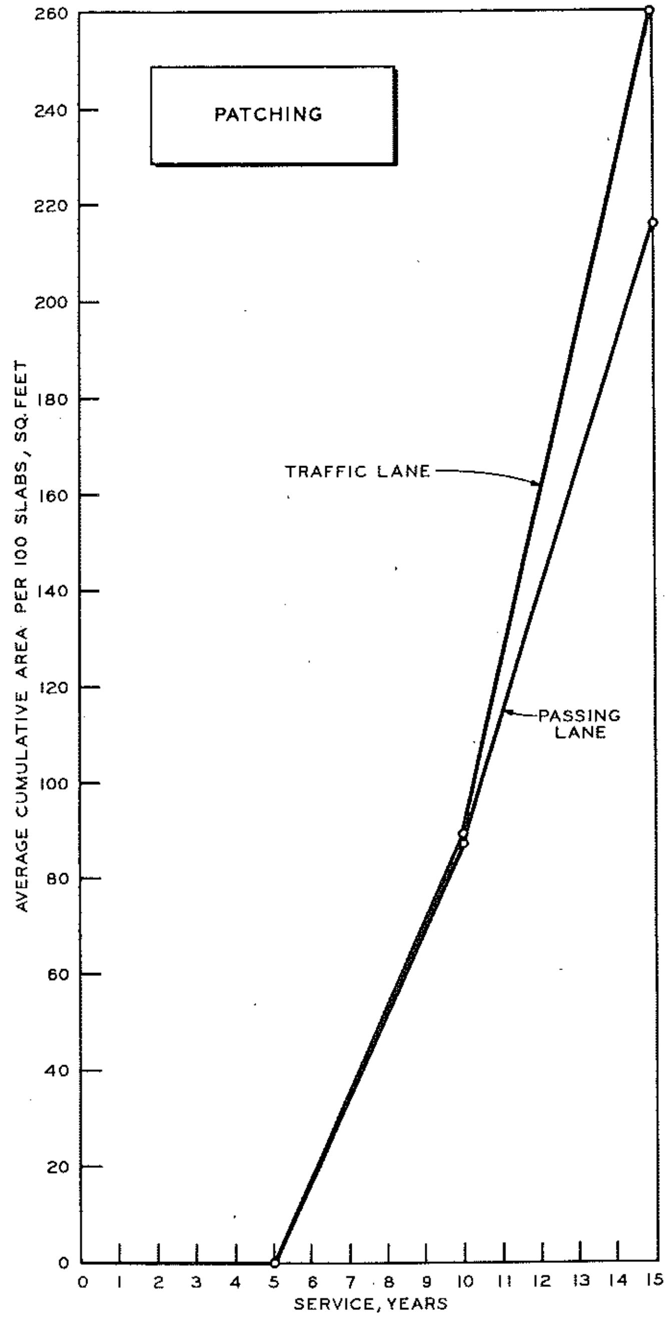

Figure 59. Effect of traffic and amount of patching.

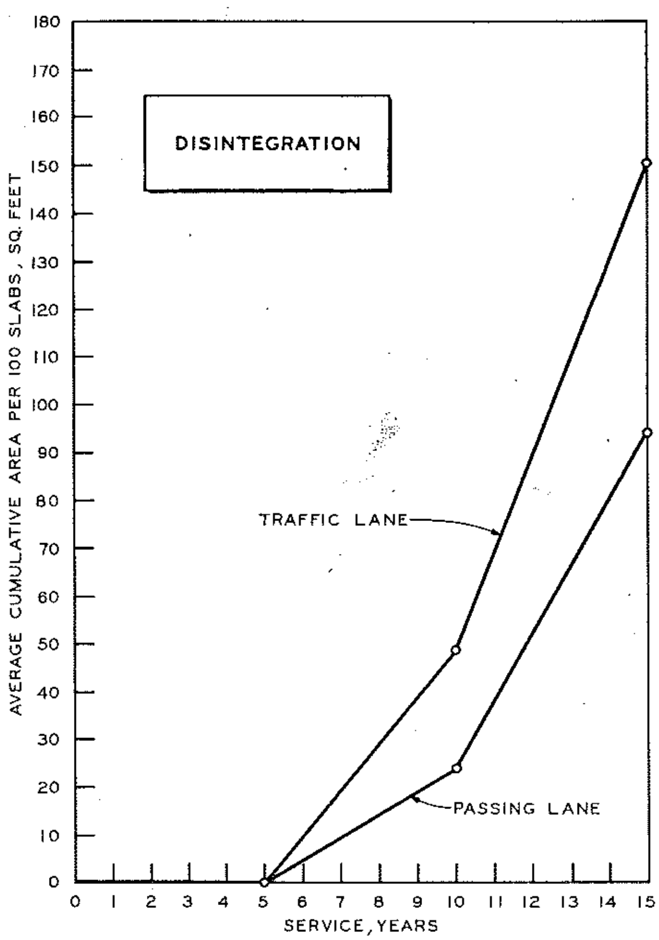

Figure 60. Effect of traffic on amount of pavement deterioration.

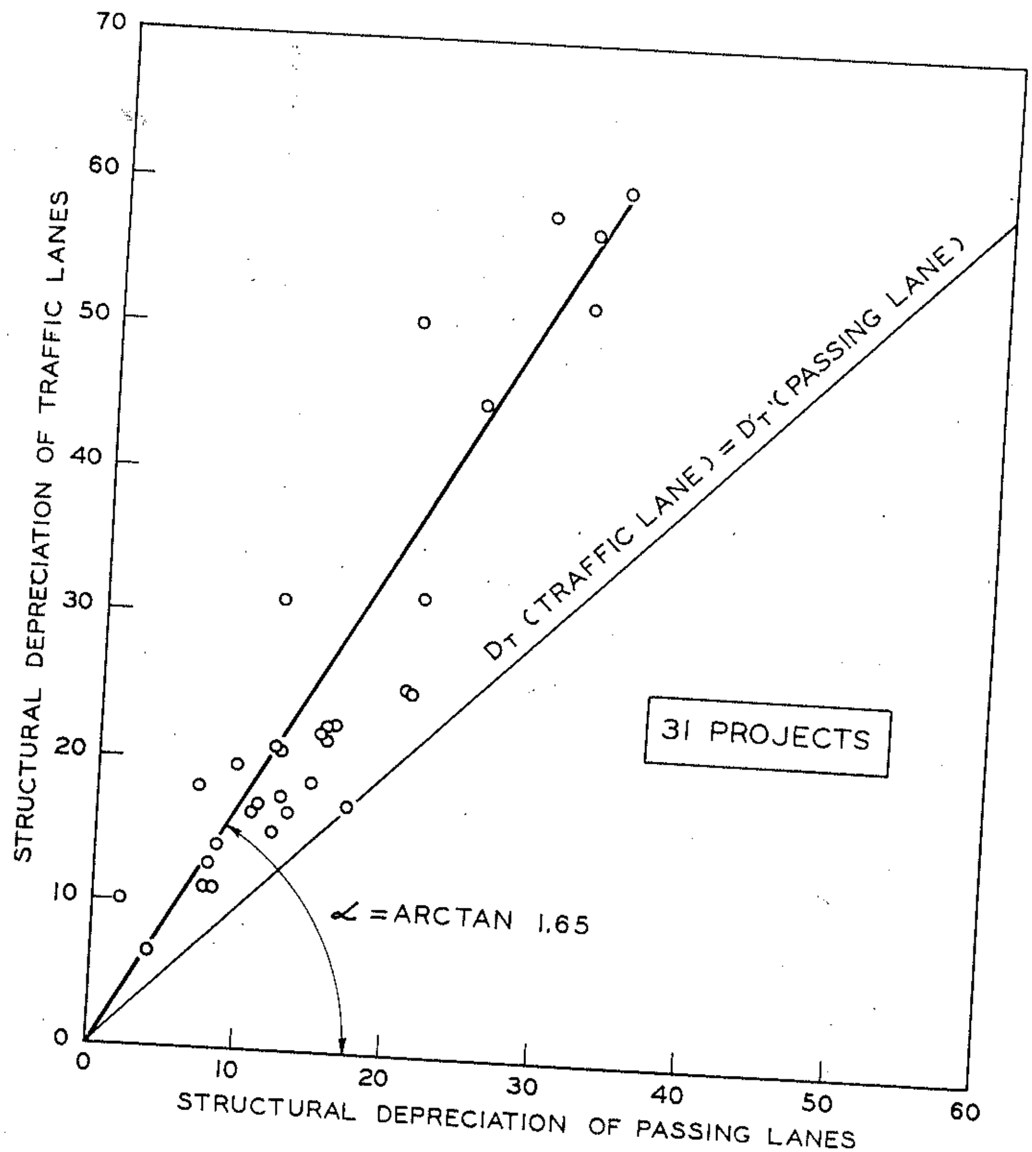

Figure 61. Average structural depreciation of passing lanes versus traffic lanes.

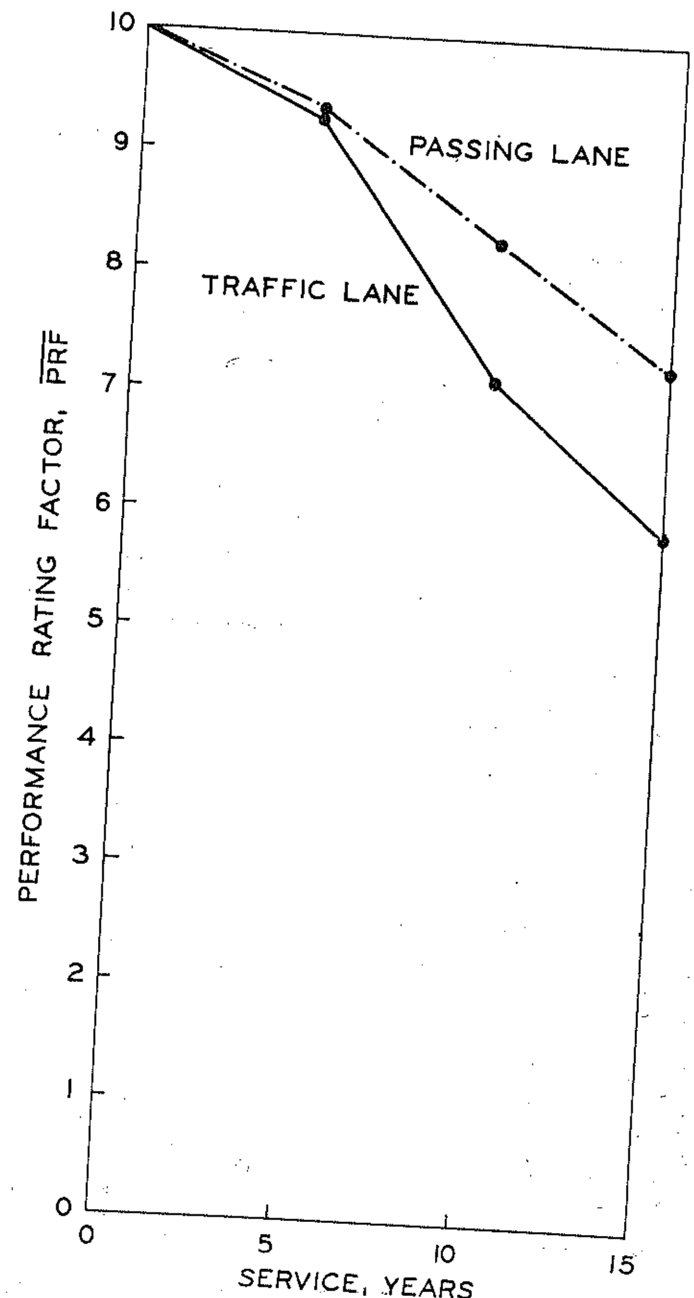

Figure 62. Performance of divided

Figure 61 shows the Depreciation Index (D) value for the traffic and corresponding passing lanes for each of 31 divided roadway projects (to be derived later). Also shown is a 45-degree line which, of course, indicates equal depreciation for each lane. The traffic-passing lane structural depreciation relationship, while linear, definitely does not have a slope of 1.0, necessary if both lanes performed equally. Overall, the traffic lanes show about 65 percent more structural deterioration (as measured by D) than the corresponding passing lanes. Figure 62 shows the same performance difference as measured by PRF.

As with overall traffic volumes, PSI's for matched traffic and passing lanes do not show appreciable differences even after fifteen years of service (Fig. 63). As could be expected, the survey variables which are heavily weighted in the structural performance index; i.e., corner cracking and spalling together with transverse and longitudinal cracking, do not noticeably influence PSI in the traffic wheel paths where roughometer measurements are made. Only after more extensive deterioration has taken place (much of which may stem from the initial failures measured by these variables) would roughness measurements be substantially affected. Because of the failure of the PSI to adequately reflect structural performance differences, further analysis with this measure was not undertaken.[5]

It should be pointed out that all projects herein examined were considered to have adequate subgrade support. Either they were constructed on natural sand and gravel subgrades with good natural drainage, or proper subbases with adequate thickness constructed to improve drainage. If performance is measured by the PSI, one would conclude from the traffic- passing lane study, that since commercial traffic does not perceptibly influence structural performance, subgrade support is adequate (Fig. 63). However, if performance is measured by structural condition variables collectively in indices such as D, one would conclude the opposite: because pavement performance, even with quality subgrade, is substantially affected by commercial traffic; grade support together with construction procedures could be improved.[6]

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[5] All survey variables in equation 1 except blowups had larger average values in the traffic lane.

[6] Since these projects were constructed, the Michigan subbase requirements have increased from 12 to 14 and thence to 15 in. of granular material. Also, contraction joint spacing has been decreased from 99 ft to 71 ft 2 in.

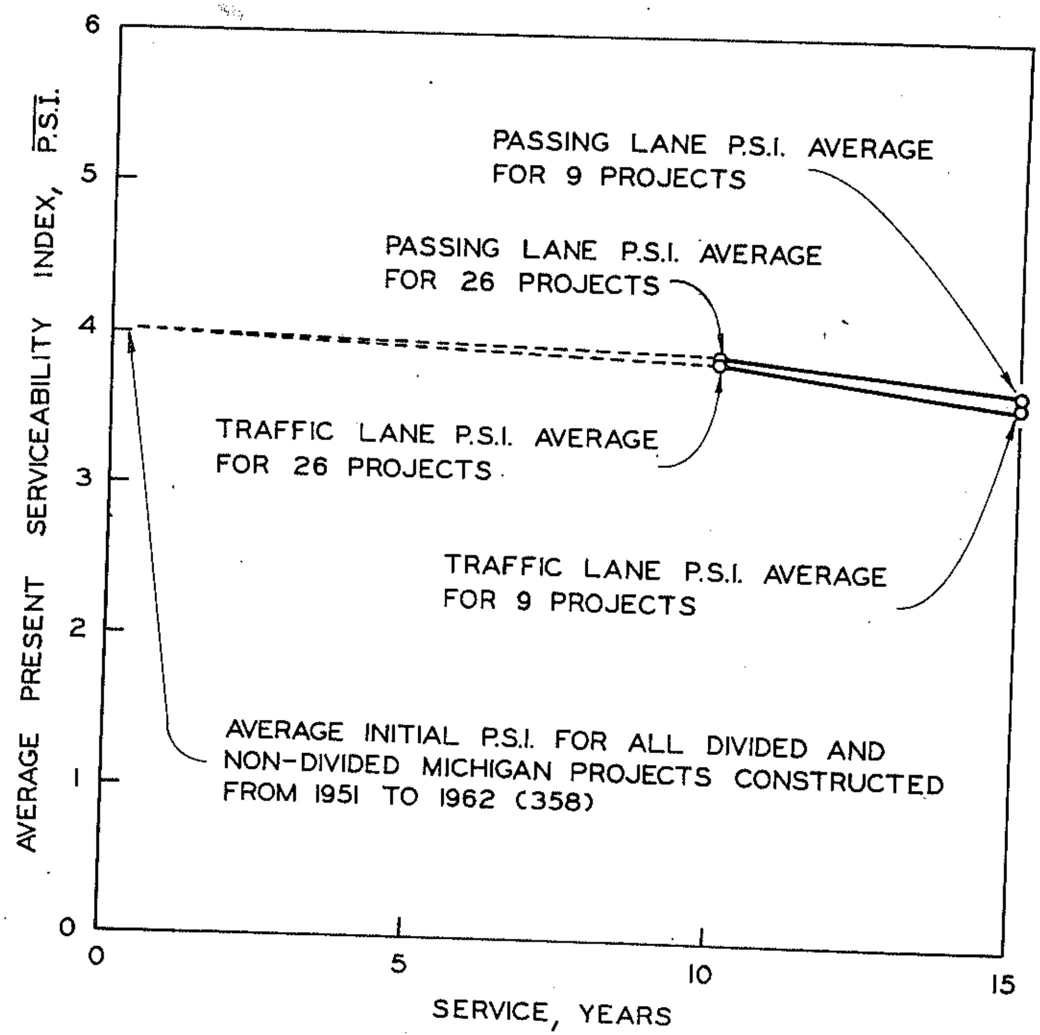


Figure 63. Present Serviceability Index by lanes.

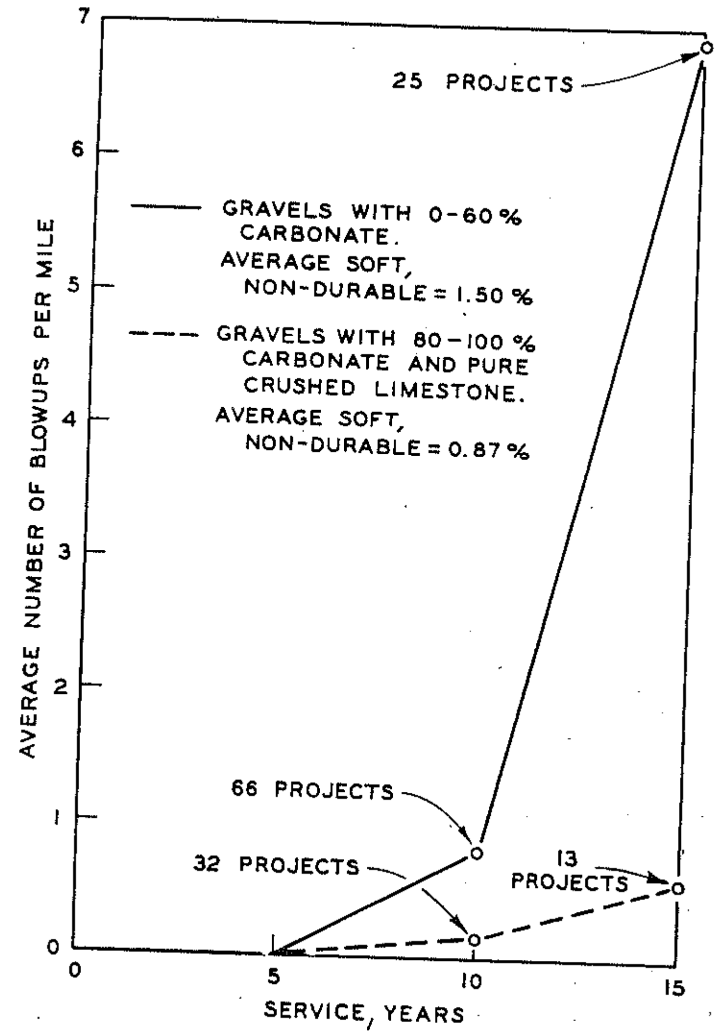


Figure 64. Blowup performance for two classifications of aggregate.

These examinations suggest three conclusions:

1. Of the deleterious materials examined, soft, non-durable content is the best predictor of ten- to fifteen-year structural performance as measured by either subjective or objective indices.

2. Average daily commercial traffic is definitely associated with structural deterioration as measured by the same indices. This is shown to be the case for both two-lane and four-lane divided roadway.

3. PSI does not appear significantly related to structural performance. Also, neither soft, non-durable content in coarse aggregates nor commercial traffic volume show any relationships to PSI. This may not be true for service periods beyond fifteen years when structural deterioration substantially affects roughness. However, up to 15 years of service, condition survey variables combined in indices appear to be the only effective early predictors of general structural deterioration.

Performance of Individual Variables - Blowups

One basic variable of special interest and concern is blowups. Because this form of pavement distress is more infrequent and occurs later in pavement life than most other kinds, it is more difficult to analyze. Blowups have been attributed to coarse aggregate types because of unusually high frequencies found in pavements constructed from aggregates coming from specific pits (14). Figure 64 shows substantial blowup incidence for gravels containing 0-60 percent carbonates. It should be noted that the difference in blowup performance itself for the two aggregate groups is a function of time--no difference at five years; some difference at ten years; and considerable difference at fifteen years.

More intensive examination of the aggregate data shows that blowup performance is probably related more to aggregate heterogeniety than carbonate content. For the present case, heterogeneity (H) was previously defined as:

$$H = \sin (P \pi)$$

where P is the estimated proportion of carbonate in the aggregate.

Figure 65 shows the relationship between blowups and aggregate heterogeniety (H) for four carbonate classes: 100 percent, 80-90 percent, 40-60 percent, and 0-35 percent. After fifteen years of service, homogeneous aggregates (100 percent crushed limestone or dolomite having an H value of zero) show very few blowups while highly heterogeneous aggregates (40-60 percent carbonates having an H value of 1.0) show substantially increased blowup frequency. As with general performance indices, blowups could be attributed to differential thermal expansion rates of soft, non-durables (15). Since Michigan specifications already control for soft, non-durables, it is expedient to develop models and make decisions on this basis (6, 16, 17).

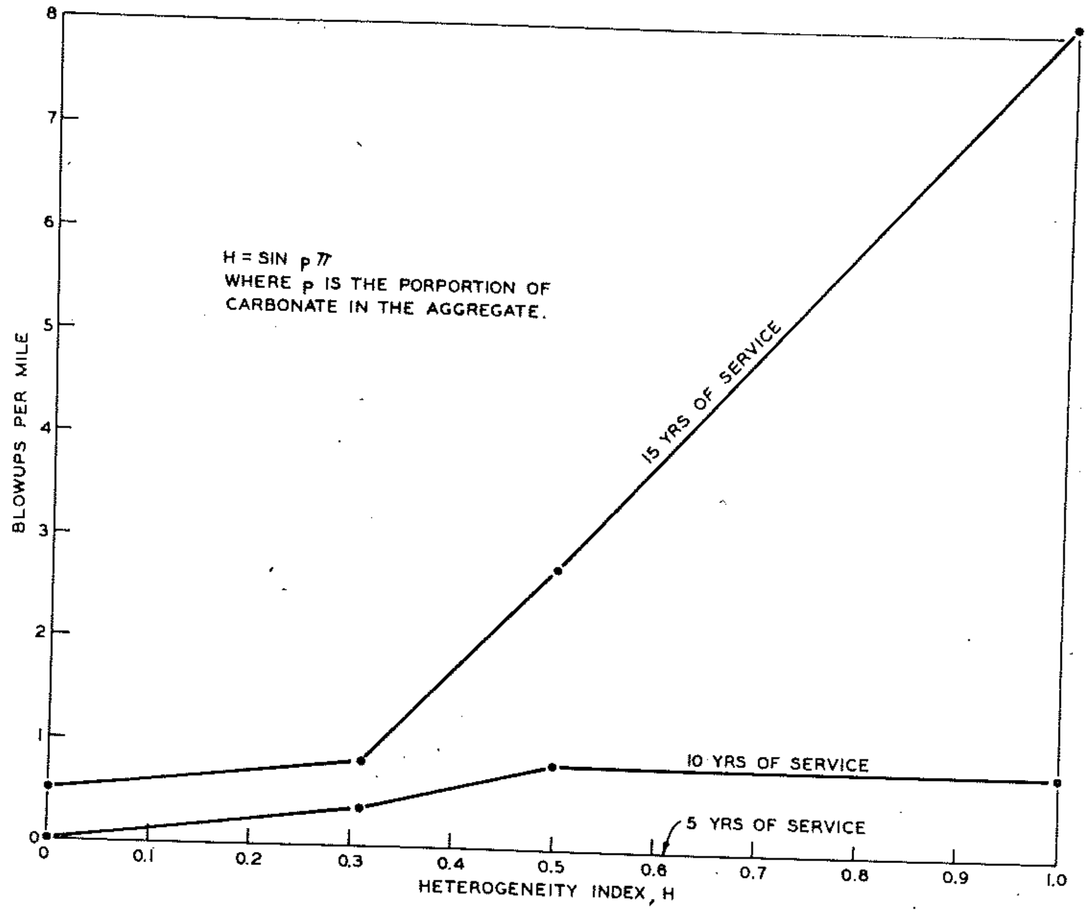


Figure 65. Effect of coarse aggregate heterogeneity on blowups.

Structural Performance Model

Assuming that the effects of soft, non-durable particles and average daily commercial traffic volume on performance are real, it seems reasonable to construct a performance prediction equation: Figures 46, 49, 52, and 53 suggest the following relationships:

$$\log \hat{I} = K_1 \log S + \log K_2 \quad (4)$$

$$\log \hat{I} = K_3 \log (ADCT) + \log K_4 \quad (5)$$

$$\log \hat{I} = K_5 \log t + \log K_6 \quad (6)$$

where:

$\hat{I}$ = estimate of structural deterioration index (I)

A = percentage of soft, non-durable in coarse aggregate

ADCT = average daily commercial traffic volume

t = Age of pavement in years

$K_1 \ldots K_6$ = Fitting constants with K_2 and K_4 dependent on the survey year. Ideally, these variables would be combined by multiple regression techniques. In the present case, however, a complete data matrix was not available since many projects did not have complete records for soft, non-durable content or condition surveys for all three service periods. Consequently, these variables were combined geometrically by first estimating the slopes for each variable by simple linear regression, and then summing the three equations (the geometric average results in a simpler model than the arithmetic average). Thus, we have in general:

$$\log \hat{I} = \frac{K_1}{3} \log S + \frac{K_3}{3} \log (ADCT) + \frac{K_5}{3} \log t + \log K_7$$

and in the present case, since $K_1 = K_3 = 0.54$ and $K_5 = 1.56$,

$$\hat{I} = K_7 S^{.18} (ADCT)^{.18} t^{.52}$$

Estimates of D_T can be obtained by integration:

$$d\hat{D}_T = \int_0^T I\, dt = K_7 S^{.18} (ADCT)^{.18} \int_0^T t^{.52}\, dt$$

and

$$\hat{D}_T = K_8 S^{.18} (ADCT)^{.18} T^{1.52} \quad (7)$$

In order that both traffic and passing lanes of divided expressway could be included with the two-lane pavements, ADCT given for the roadway only had to be distributed between the lanes. An approximate distribution formula was obtained as follows by equation(5)and the slopes in Figure 49:

$$\hat{I} = K_4 \text{ (ADCT) } K_3 \text{ and } \hat{D}_T = \int_0^T \hat{I}\, dt = K_4 \text{ (ADCT)}^{K_3}\, T$$

Dividing D_T for the traffic lane by D_T for the passing lane, and substituting the slope from Figure 49, we have:

$$\frac{\hat{D}_T \text{ (Traffic Lane)}}{\hat{D}_T \text{ (Passing Lane)}} = \left(\frac{\text{ADCT Traffic Lane}}{\text{ADCT Passing Lane}}\right)^{.54} \quad \alpha = \text{arc tan } \alpha = 1.65$$

from Figure 61. Therefore:

$$\frac{\text{ADCT traffic lane}}{\text{ADCT passing lane}} = (1.65)^{\frac{1}{0.54}} = 2.52$$

Because the percentages in the traffic and passing lanes must add to 100, we have:

$$\text{ADCT traffic lane} = .72 \text{ ADCT}$$

and

$$\text{ADCT passing lane} = .28 \text{ ADCT}$$

These formulas were used to divide the total roadway ADCT into volumes for each lane, thereby permitting the incorporation of both lanes of divided expressway into the final analysis.

K_8 of equation 7 can now be determined by regression of $\hat{D}_T$ on D_T , i.e.,

$$\hat{\hat{D}}_T = K_9 \ \hat{D}_T + K_{10} \qquad \text{which gives}$$

$$\hat{\hat{D}}_T = .33 \ S^{.18} \ (ADCT)^{.18} \ T^{1.62} - 28 \qquad (8)$$

The correlation coefficient for the regression is +0.77 which for the number of points (111) is highly significant.

Figure 66 is a plot of D_T and $\hat{D}_T$ for all projects for which five-, ten-, and fifteen-year data were available including divided expressways.

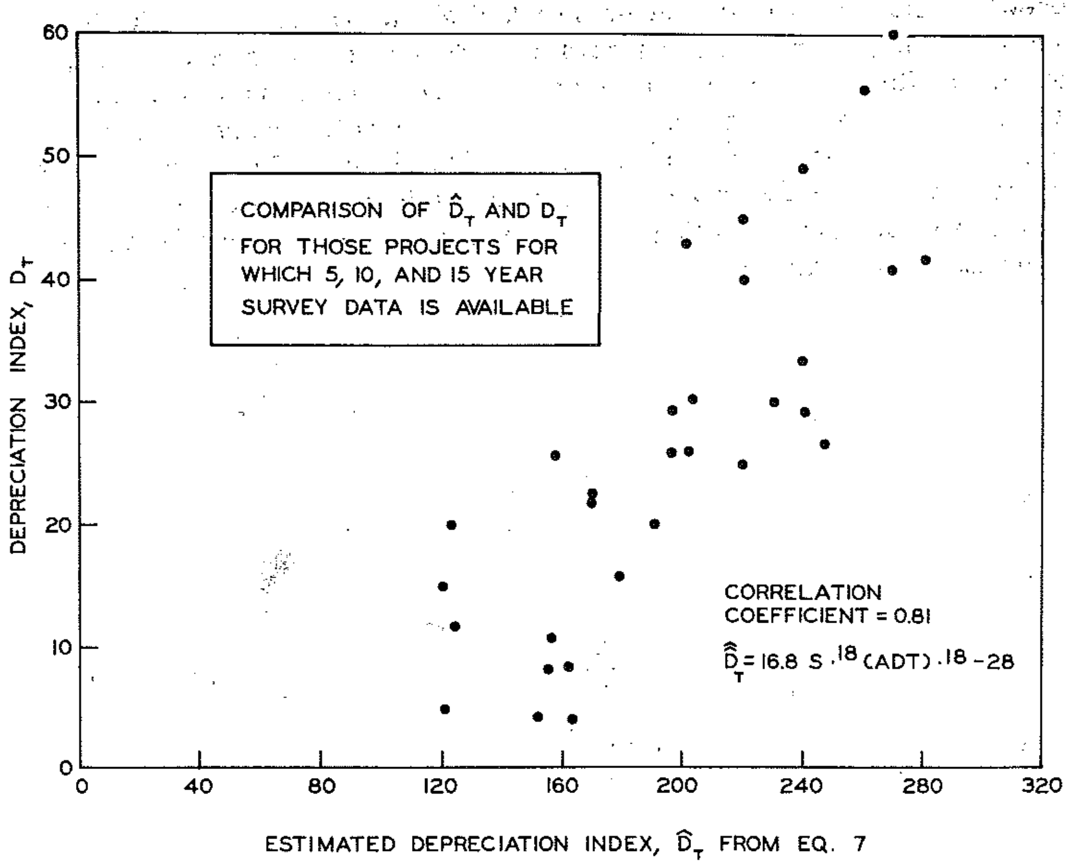


Figure 66. Comparison of $\hat{D}_T$ and D_T for those projects for which 5-, 10-, and 15-year survey data are available.

This is presented to show the structural depreciation predicting power of S and ADCT independently of time, when reasonably stable (three survey years available) D_T values can be calculated. Considering only these projects, equation (7) becomes:

$$\hat{\hat{D}}_T = .28 \ \hat{D}_T - 28 = 16.8 \ S^{.18} \ (ADCT)^{.18} - 28$$

and the correlation for the regression of D_T on $\hat{D}_T$ is a highly significant +0.81 for 31 projects. Thus, when only five-, ten-, and fifteen-year surveys are considered, 65 percent of the structural depreciation variance can be "explained" by the variables of soft, non-durable percentage and average daily commercial traffic volume. Presumably, if more surveys could be incorporated, thereby allowing more reliable time-deterioration curves, a somewhat better relationship could be established. The mainpoint is, however, that both percent soft, non-durable in the coarse aggregate and commercial traffic have been shown to have structural performance prediction power. This does not *ipso facto* "prove" causal relationship. However, background information on these variables (such as traffic-passing lane comparisons and the AASHO Road Test) suggests that the present correlations are meaningful, and that these variables are not merely "standing-in" for the "real" causes.

OBJECTIVE IV

RELATIONSHIP BETWEEN SOIL AND PERFORMANCE

Subbase Materials

In the early 1930's, sand subbase was found to be effective in reducing the spring thaw breakup of Michigan pavements. Subsequently, the use of sand subbase became more widespread throughout the State until, by 1940, a 12-in. sand subbase was required under all rigid pavements constructed over heavy natural soils. Incoherent subbase materials were stabilized through the top 3 in. with salvaged or pit run gravel, loamy soils or, as a last choice, clay soils.

In 1955, the subbase thickness was increased to 14 in., more to facilitate construction control than to improve pavement strength. The upper 3-in. layer of the subbase consisted of selected gravel. This 3-in. layer of gravel was to reduce losses in density due to drying and rutting, to provide a stable surface for pavement forms and paving equipment, and to prevent infiltration of fine uniform sands into the pavement joints. In 1963, the thickness of the layer of selected gravel subbase was increased to 4 in. making a total subbase thickness of 15 in.

From the history of Michigan subbase construction, it appears that subbases used under postwar rigid pavements were relatively uniform and that any relationship between subbase and pavement performance would be primarily caused by control of soil density during construction. However, subbase soil density data were not available on a project-by-project basis and thus could not be considered as a variable to study in relation to pavement performance.

Analysis of Influence of Subgrade Soil

Figure 42 is a map locating soils of varying suitability as subgrade materials. Figure 42 was developed from a Soil Map of Michigan by J. O. Veatch delineating soil areas for agricultural use. However, Veatch's map showed approximately 100 different soils which, for this study, were grouped by suitability for subgrade materials into three categories, "good," "fair," and "poor." Properties of the soils considered were drainage together with wet and dry strength.

In order to determine if these categories conveyed information on structural performance, the average transverse crack incidence for each of these categories over the fifteen-year service period was plotted in Figure 67. Notice that the order of performance follows soil quality for the 125 projects surveyed. It should also be remembered that the average for the fifteen-year service periods is less than would be expected. This is be-

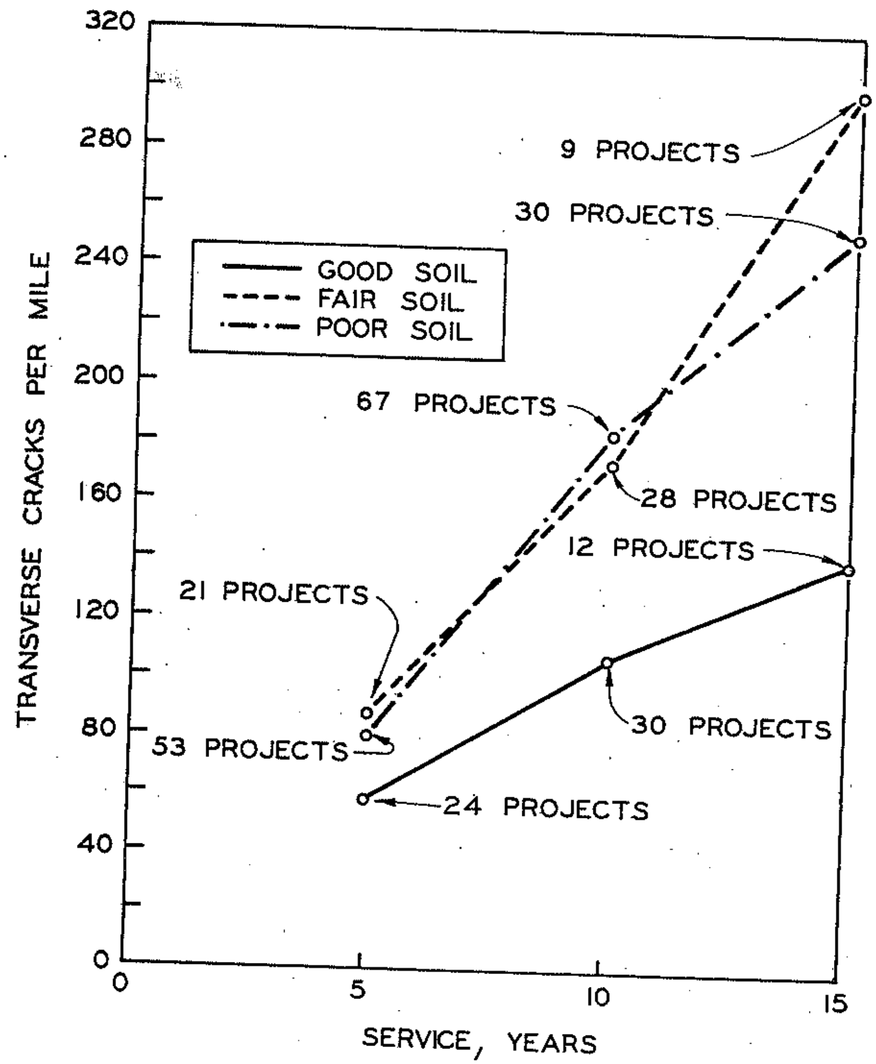


Figure 67. Soil versus transverse cracks.

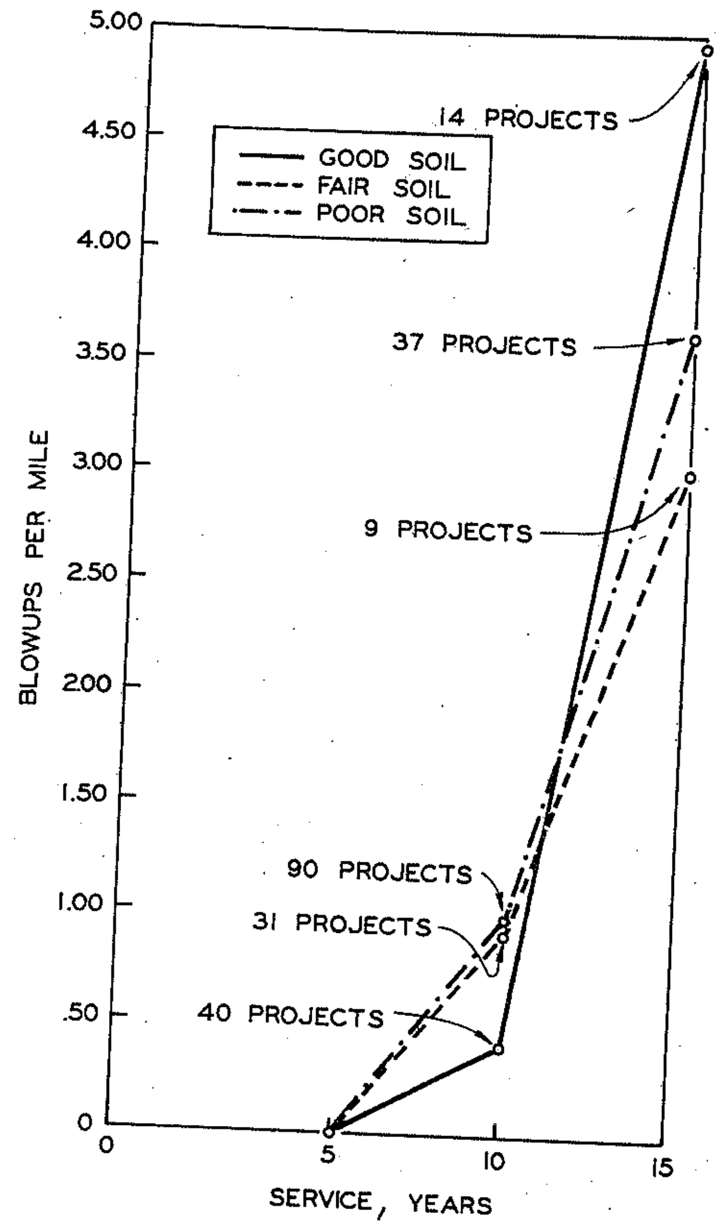


Figure 68. Soil versus blowups per mile.

cause highly cracked portions of ten-year projects are generally resurfaced before fifteen years and this results in substantial underestimation of fifteen-year cracking. Even though resurfaced portions are not considered in the analysis, the biasing effect is still present.

Because soil quality regions are somewhat geographically correlated with aggregate quality, it was decided to compare soils groups using variables known not to be affected by subgrade support. To this end, blowup frequencies for each soils group were compared both graphically and statistically (Fig. 68). As expected, no association of blowups and soil quality was found. Therefore it was assumed that coarse aggregates were not responsible for the pavement performance differences in question.

The only other variable found to be related to general performance was average daily commercial traffic volume. Figure 69 shows that transverse cracks are associated with commercial traffic. Moreover, the association is stronger with age: $r = 0.14$, 0.33, and 0.52 for the five-, ten-, and fifteen-year surveys, respectively. The latter two correlations are significant and highly significant. It was thought that the cracking-traffic correlation might be responsible for the findings concerning the soils (Fig. 67) since it was known that many generally poor soil regions in Michigan are found in high population density areas; just as good soils are often found in remote regions with low traffic volumes. To show the statewide relationship between soils and traffic, the average commercial volume for each soil type was plotted in Figure 70. Despite considerable scatter in the raw data, a trend is apparent: pavements built in regions with poor soils generally have higher commercial volumes than those built in regions of good soils. Because of this fortuitous correlation, the original findings; namely that a geographic soils map indicated performance differences attributable to soil quality, is probably erroneous. The evidence, which includes the traffic - passing lane comparisons of divided expressway, suggests that structural performance in general, and transverse cracks in particular, are causally related to commercial traffic volume. Soil quality is undoubtedly an additional factor, but information based on general classifications for large geographic areas is not sufficiently precise to improve performance prediction. For example, the fifteen-year multiple correlation of transverse cracks on soil and traffic is 0.54, while the partial correlation with traffic is 0.51. Thus, we see that the addition of this type of soils information adds little to our performance forecasting ability.

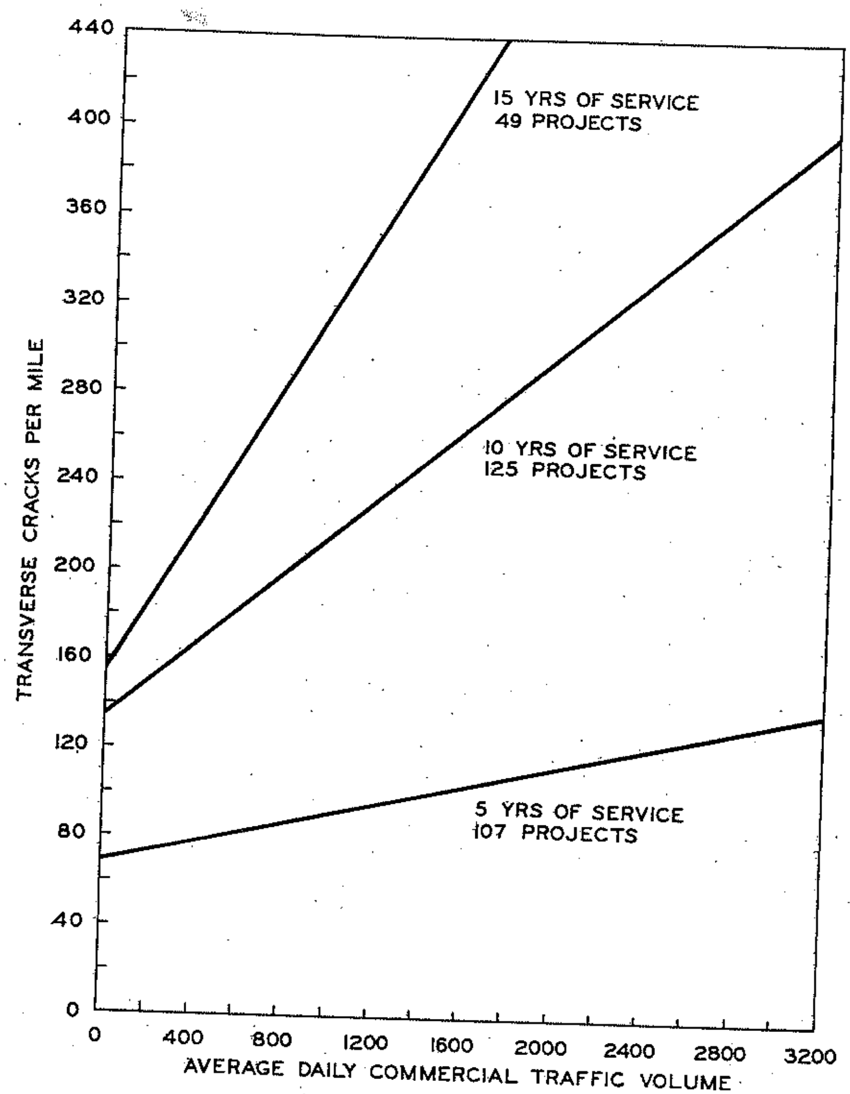
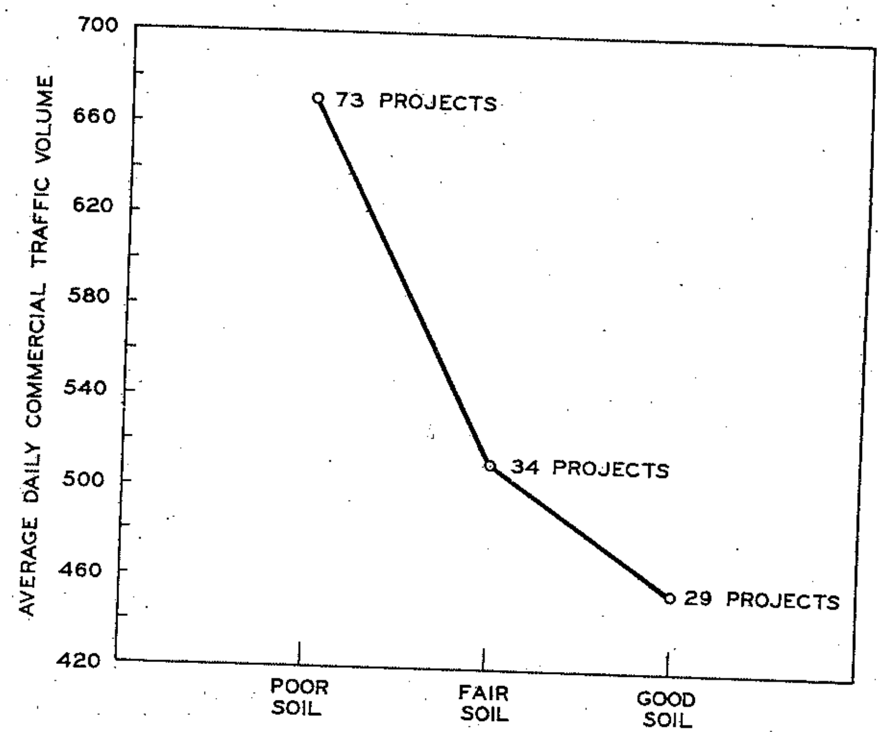


Figure 69. Traffic versus transverse cracks.

Figure 70. Soil versus commercial traffic volume.



OBJECTIVE V

DETAILED INVESTIGATION OF SPECIFIC TYPES OF PAVEMENT DISTRESS

The fifth objective of this study, to investigate in detail the causes of certain specific types of pavement distress, was less thoroughly covered than the previous objectives, primarily due to the fact that experienced engineers required to make such investigations were not available to make extensive field surveys of the best and poorest performing projects because of other parallel research efforts. The 10 best and 10 poorest performing projects for each type of pavement deterioration as noted in condition surveys were tabulated. However, for clarity only the three best and three poorest performing projects are shown in Figure 71. For some pavement deterioration categories the three best projects are not shown on the map since there were more than three which had none of the pavement deterioration under study. A detailed study of the location throughout the state of the poorest projects in terms of individual performance variables, such as transverse cracking, longitudinal cracking, etc., did not elicit any unusual distribution of poor projects.

Field investigation for each very poor and very good performing projects for each condition survey variable may have disclosed some casual reasons for performance differences. However it is apparent from previous investigations that only certain causes for poor performance are disclosed by these, so called, "post-mortem" examinations. For example, on a project where there is extensive longitudinal cracking, the depth of the saw-cut which forms the longitudinal plane-of-weakness can be measured and a comparison can be made between depths of saw-cuts in pavement areas, with and without longitudinal cracking. If there is a marked reduction in depth of saw-cuts in the area where longitudinal cracking occurred, it may be concluded that this deficiency resulted in the increased longitudinal cracking. However, if the contractor's equipment inadvertently passed over the pavement prior to the forming of the longitudinal saw-cut, and established a fine crack--perhaps invisible at that time--and this crack thus functioned in place of the subsequent saw-cut as a plane-of-weakness for relieving transverse warping of the pavement due to temperature and moisture changes, then this cause for excessive longitudinal cracking would never be disclosed by later field inspections. From the possible spectrum of causes for poor pavement performance, only a small percentage can be obtained by examination of routine material and construction records and subsequent field inspections after the poor performance is noted.

The Research Laboratory has had the responsibility, upon request of other Department Divisions, to investigate and attempt to determine the cause of certain specific weakness in performance of some individual construction projects. As a result of these special investigations, certain cause and effect relationships on individual projects have been established.

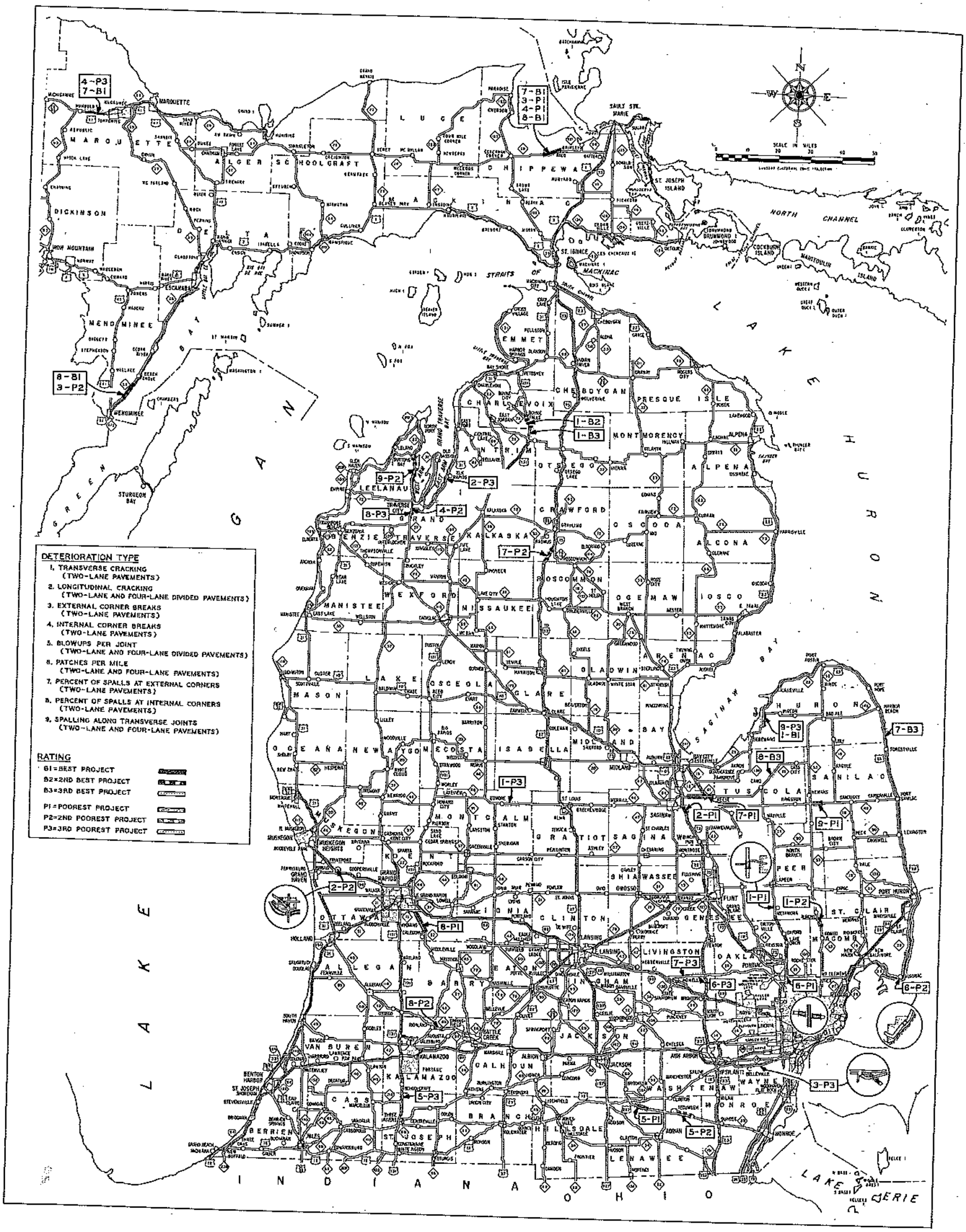


Figure 71. Location of best and poorest performing projects for 9 types of pavement deterioration (10 years of service).

It is difficult, however, to generalize from these specific investigations; therefore the causes for a specific type of poor performance for an intensively studied project may or may not be the cause of poor performance for a number of other projects exhibiting similar types of poor performance.

Since it was not possible to make a broad and extensive--and at the same time intensive--study of each type of pavement distress, some of the intensive investigations on specific projects will be summarized to indicate the causes for poor performance in specific instances.

Joint Failures

Early in 1951 serious concrete joint failures were observed on four joints on a construction project built in 1947 on M 59. Nothing conclusive concerning the cause of this joint problem could be obtained by the usual observational inspection and a condition survey of the construction project. In 1952 an expansion joint was opened in an attempt to determine the cause.

The examination of the joint disclosed the following conditions:

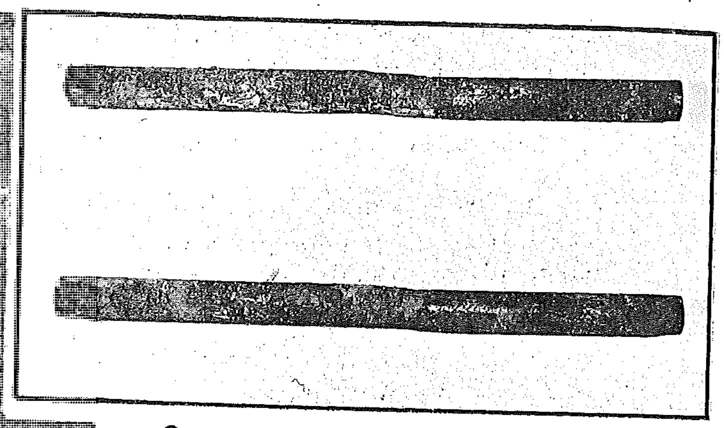
1. The metal expansion caps, installed on the end of dowels to permit pavement expansion, were never installed. Therefore, the compressive force on the end of the dowel shattered the concrete and bent the bar (Fig. 72).

2. Some of the dowels were badly misaligned both horizontally and vertically.

3. The load transfer assembly on the westbound lane was placed higher than the assembly on the eastbound lane. Consequently the top of the dowels to the pavement surface averaged 2-1/4 in. in the westbound lane and 3 in. in the eastbound lane. When properly positioned the dowel should be centered vertically in the 8-in. pavement. Thus, this distance for 1-in. diameter dowels should be 3-1/2 inches.

4. The dowels were badly rusted and pitted in the vicinity of the expansion joint filler. The reduction in dowel diameter due to rusting was approximately nine percent for the five-year life on this pavement joint.

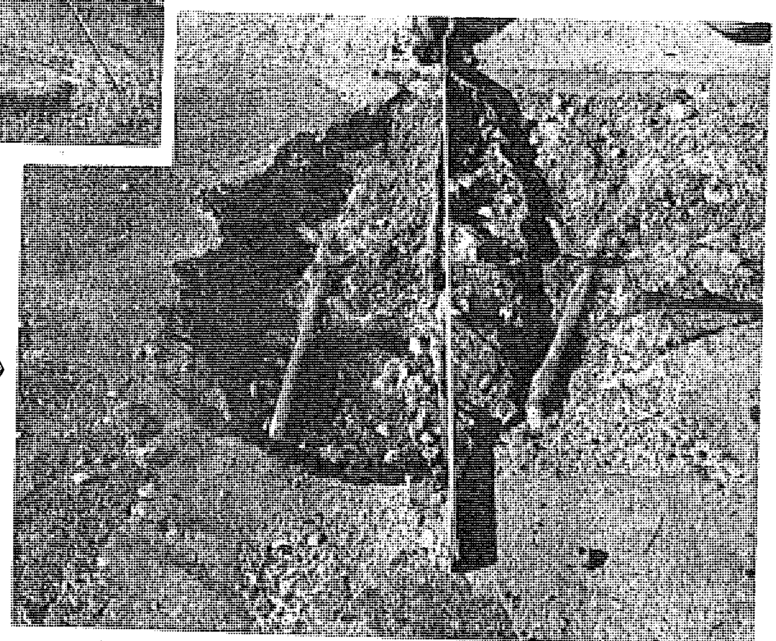
There were also contraction joints on this same project that showed distress but it was not possible to examine them since maintenance forces had repaired them before we were notified of the problem. Of the four observations noted for the expansion joint, the second and third (dowel misalignment and the dowel assembly too high) could cause similar distress for contraction joints.

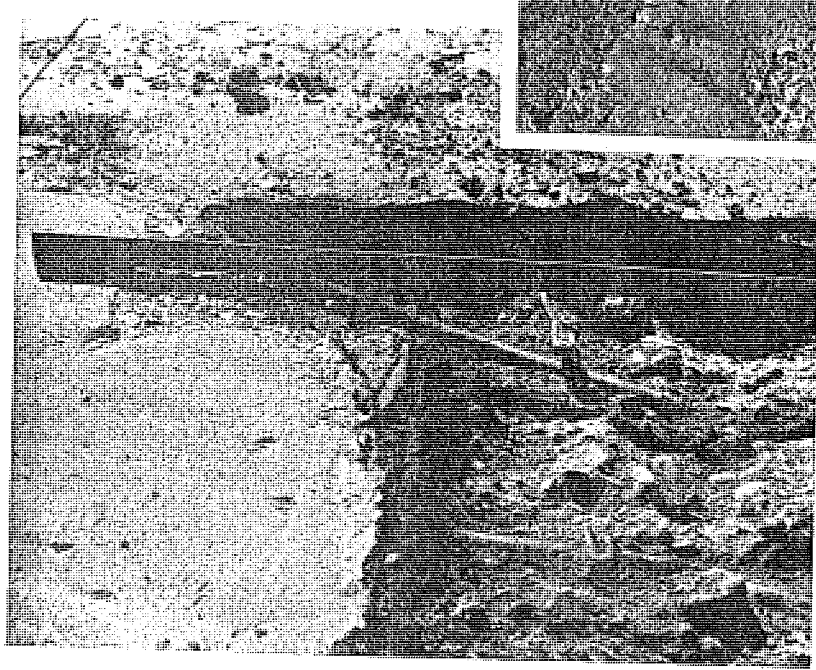


Physical condition of two dowels; note bending due to compressive force.

Condition prior to opening joint.

Horizontal misalignment of two dowels next to longitudinal center joint.





Vertical misalignment due to dowel slipping out of dowel assembly clip.

Figure 72. Joint failure and condition of dowels after exposure of joint.

In addition, a review of pavement core data indicated that the pavement thickness averaged approximately 1/4 in. thinner than the 8 in. thickness specified.

The somewhat thinner pavement, together with a large volume of commercial trucks hauling gravel from local pits to Detroit by this route, a combination of possible high positioning of the dowel bars, and possible dowel misalignment at some transverse joints appear to have caused these premature joint failures.

In July 21, 1955 a blowup occurred on M 47 on 8-in. uniform thickness pavement built in 1949. Subsequent condition surveys indicated the pavement to be in excellent physical condition with an unusually low percentage of transverse cracks and spalls. This blowup was the first major physical defect to appear in this project after six years of service. After extensive investigation, the cause of this blowup was not definitely determined but it was strongly suspected that low quality concrete in that particular area might have been a primary factor in the incident. This observation was based primarily on the fact that in the spalled areas of the joint the separation was almost entirely between coarse aggregate and mortar with very little fracture of coarse aggregate, indicating that the binding properties of the mortar was the weakest link in the aggregate-mortar system. Again, dowel bar corrosion resulted in a net reduction in bar diameter after six years of service of between 3 and 10 percent for these 1-in. dowels.

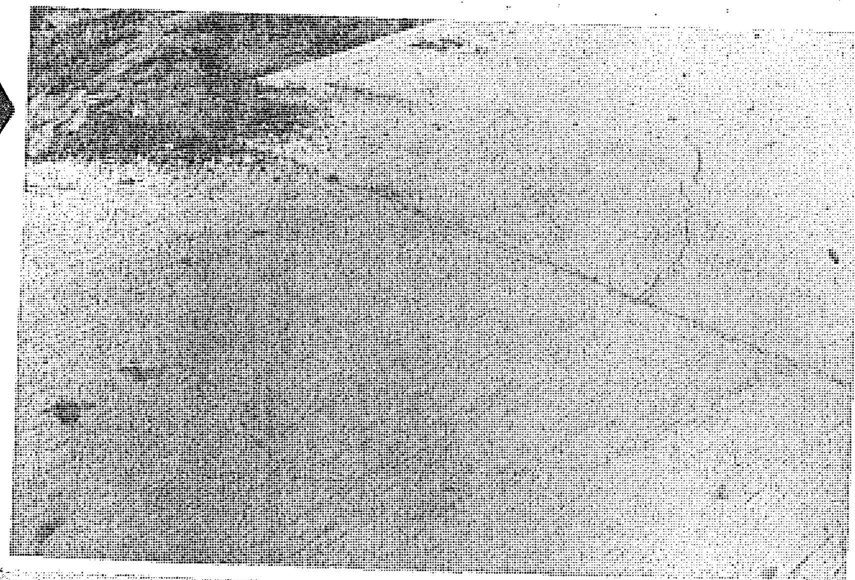
In 1959 the Research Laboratory summarized joint problems that had developed over the past few years. These included: 1) longitudinal cracking at the joint, 2) joint spalling, 3) joint blowups, 4) inadequate load-transfer assemblies, 5) dowel bar corrosion, 6) concrete failure at construction joints, and 7) inadequate joint sealing (Figs. 73 and 74).

The causes for these problems were outlined in this report. Some of the reasons for longitudinal cracking were:

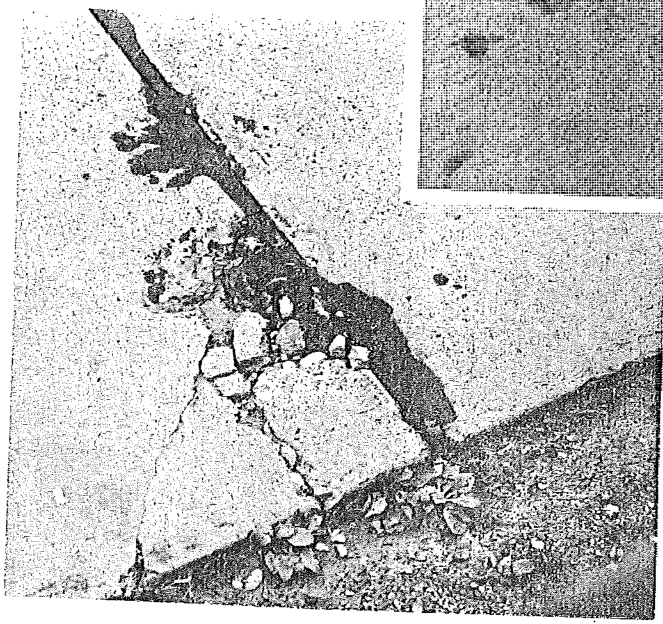
1. Heavy loads during early life of the structure, such as earth-moving machinery or other heavy contractor's equipment. (This is especially true for sawed longitudinal joints in comparison with the previous practice of a premolded bituminous strip to form the longitudinal plane-of-weakness joint, since any heavy equipment on the slab, particularly when in a temperature-warped condition prior to forming the longitudinal joint by sawing is a very effective means of causing a longitudinal crack.)

2. Uneven subgrade support, particularly loss of pavement edge support which may lead to longitudinal cracking.

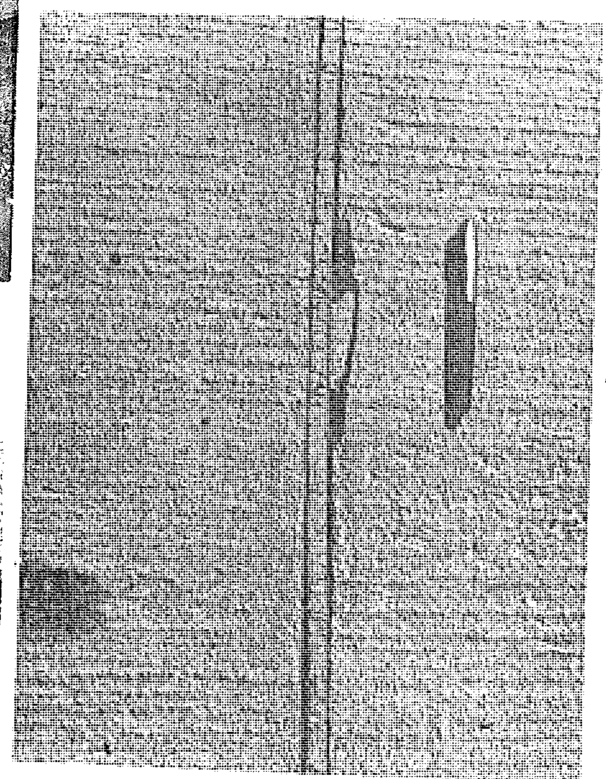
Longitudinal cracking at the joint.



Concrete spalling; external corner.



Joint groove spalling.



Concrete spalling; interior corner.

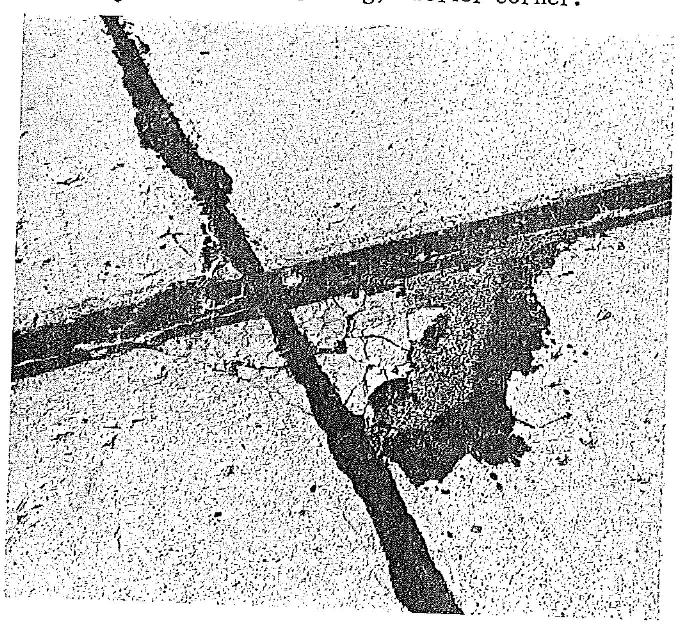
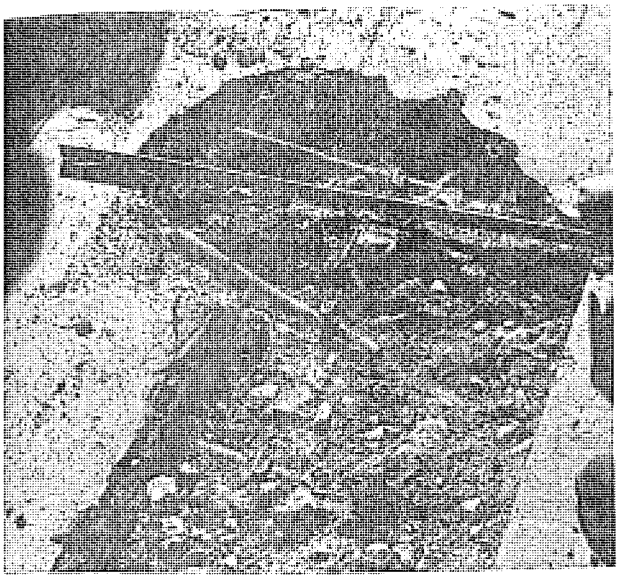
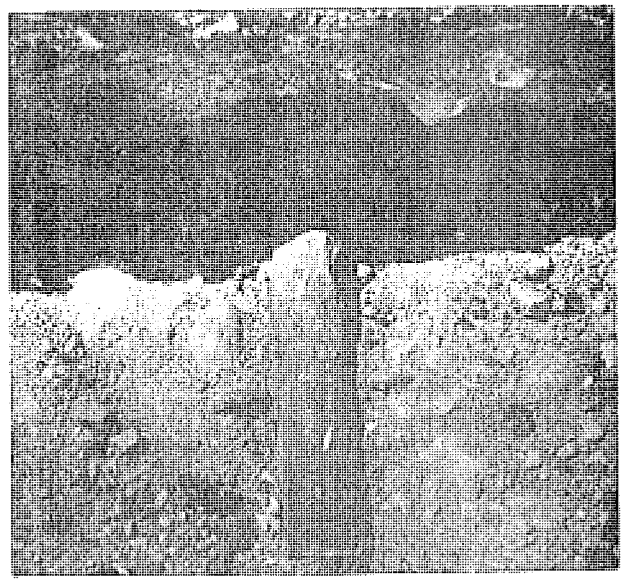


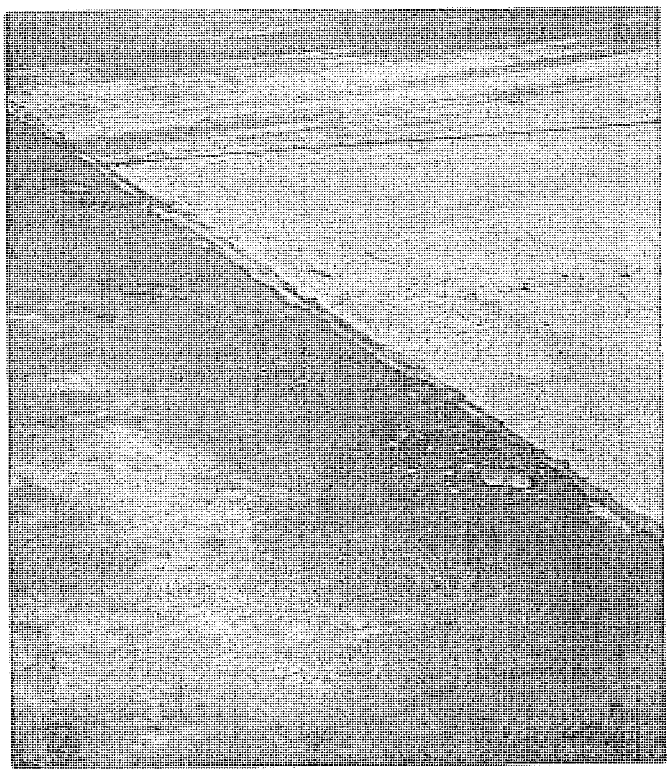
Figure 73. Two joint problems. Longitudinal cracking and joint spalling.



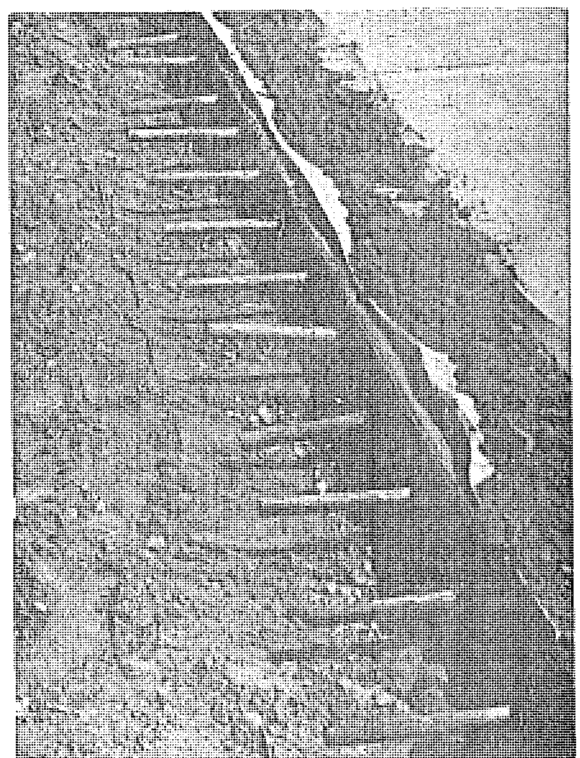
Dowel misalignment during concrete placement due to inability of assembly to hold dowels in place.



Corroded dowels in place in 9 year old pavement.



Difference in appearance and performance of concrete at each side of construction joint.



Misalignment of dowels at construction or night joint.

Figure 74. Joint problems--dowel misalignment, dowel corrosion, and inferior concrete at night joint.

3. Infiltration of inert soil particles from the shoulder causing unusual transverse joint facial pressure.

4. Misalignment of dowels in the pavement joint.

5. Localized pressure at slab ends caused by unequal volume changes due to moisture variations in the slab width.

6. Frozen dowel bars at joints caused by rusting and lack of proper lubrication for expansion joints.

Two types of joint spalling were considered, 1) spalling of the joint groove; generally this type of spall extends only the 2 in. depth of the joint groove, and the crack is quite close to the joint (within 1 in.), and 2) spalling of the joint face, which is most prevalent at the exterior or interior corners of the slab and is illustrated in Figure 73. The joint groove spalling problem in current construction has been greatly attenuated by several factors. Rather than forming the joint groove by means of a removable mandrel or by placing styrofoam to form the groove and subsequently removing it, the present joint grooves are formed by sawing. Secondly, since preformed neoprene joint seals are being used in place of hot-poured rubber-asphalt joint seal, any spalling of the joint groove at the time of construction must be repaired to obtain a proper joint groove face for the neoprene seal. Epoxy mortar has been used to effect these spall repairs.

We feel that the primary cause of joint spalling is the infiltration of foreign material into the joint groove and the plane-of-weakness crack below the joint groove. Current observations on preformed neoprene joint seal indicate that the joint groove and the crack are being kept free of this infiltration and this should be a much less serious problem for current construction projects.

The third joint problem considered in the 1959 report was joint blowups. It was stated that these generally occur after about eight years for post-war pavements. The postponement of this problem in prewar construction for a longer period is ascribed to the use of expansion joints. However, it has definitely been established that the use of expansion joints exclusively, does not eliminate the problem but only postpones it. If infiltration of foreign material into the joint is not controlled, since we feel this is a primary cause of blowups, then the use of expansion joints simply means it takes longer to use up the storage space provided. As discussed previously, reducing joint blowup problems can be accomplished by reduction in the soft, non-durable content of the coarse aggregate in the concrete and by improving joint seal performance. In this respect we feel that the use of preformed neoprene joint seal will greatly reduce joint infiltration and thus reduce the frequency of joint blowups in the future.

The examination of many joint blowups has shown that in most cases some construction factor has triggered the failure of this particular joint. These factors include misalignment of dowels, faulty dowel baskets, inferior concrete--particularly at construction joints--faulty placement of reinforcing steel, or frozen dowels.

In examining joints that have failed it has been obvious that inadequate dowel bar assemblies played their part in the poorer performance. Early assemblies did not securely restrict the end of the dowel from displacement since they held the dowels by means of a harp-shaped clip box in which the dowel was snapped down into position. Figure 74 indicates how the force of concrete, or placing operations, have dislodged the end of the dowels from the clip and then forced one end up and out of position.

In 1953 the Department specified 1-1/4-in. dowels rather than 1-in. specified previously, and also required rigid load-transfer assemblies for holding the dowels in alignment within the assembly. Unfortunately, the number of construction projects with ten years of service with the upgraded type of load-transfer assemblies were insufficient for this study to determine if this would result in a reduction in pavement joint blowups. Since blowups rarely occur in less than ten years of service any conclusions in this respect must be delayed for a few more years.

The extent to which dowel bar corrosion may influence joint failures is difficult to determine; however, such corrosion can be expected to restrict slab movement and at least be a contributing cause for poor joint performance.

Quite often when a pavement joint is investigated due to its poor performance it is found to be a construction joint used at the end of a day's pour. Invariably the pavement surface which has deteriorated is on the side of the construction joint poured at the end of the day. This is apparently due to the fact that this concrete is inferior in quality to that placed at the beginning of the day or during regular operations throughout the day. Also, misalignment of dowels placed through a joint bulkhead is another source of trouble (Fig. 74). A more recent method of correcting this situation has been to use a complete expansion dowel-bar assembly and place the bulkhead in place of the expansion filler. By this method both sides of the dowel bars are properly supported and do not get displaced or bent.

In the 1959 report it was stated that the performance of specification rubber-asphalt joint sealers was not up to expectations. Prior to this (in 1956) an experimental project was undertaken with the cooperation of the newly formed Joint Seal Manufacturers Association (JSMA) and with all six member-companies participating. The purpose of this study was to evaluate the best sealing products of the manufacturers without regard to specifications or price. A 10-mile long concrete roadway was sealed with six different makes of each of two types of hot-poured rubber-asphalt sealer (regular type meeting Federal Specification SS-S-164 and a slightly softer grade) and five brands of cold-applied materials, as well as several products developed especially for this project by the various manufacturers. These special products included both hot-pour and two-component cold-applied materials of the jet-fuel-resistant type. In all, 24 different joint sealing materials were used. In summary, after two years of service it was agreed, after an inspection of the project by JSMA and laboratory representatives, that none of the joints in the project now appeared to be well sealed. This experiment, on the basis of 99-ft joint spacing, was rather convincing evidence that joints could not be properly sealed with liquid-type joint seals and present joint construction practices.

However, one further experimental project in joint sealing was attempted to determine if the shape factor of the joint groove, as presented by Professor Egon Tons in 1959, and somewhat shorter slab lengths would result in satisfactory joint seal performance. On the project, five groove sizes were tried, 1/2 by 1/2 in., 3/4 by 3/4 in., 1 by 1 in., 1/2 by 2 in., and 3/8 by 1/2 in. The joint grooves were formed by sawing without a filler strip to form the plane-of-weakness and with a 1/4- by 2-in. premolded bituminous fiber filler strip to establish the plane-of-weakness. In addition, three slab lengths were used, 57 ft-3 in., 71 ft-2in., and the then conventional, 99-ft slab length. All joint grooves and immediate pavement surfaces were cleaned by sand-blasting and, just prior to sealing, any loose material accumulated in the grooves was removed by a jet of compressed air. A hot-poured rubber-asphalt joint sealing compound meeting Department specifications was used to seal all transverse joints. For approximately two years sealing performance was reasonably satisfactory, but none of the combination of factors attempted provided a joint seal that could be expected to perform satisfactorily for longer than about two years. The joint seals failed by adhesion or cohesion, or lack of ductility, and foreign material could be seen infiltrating the joint seal materials.

As a result of the Department's experience over more than ten years, namely that liquid-type joint seals were incapable of properly sealing transverse joints for longer slabs, it was very receptive to the use of preformed neoprene when this material was developed several years ago. It was first experimentally installed in Michigan in the Fall of 1962. In the 1964 construction season neoprene was installed on eight construction projects. In 1965 it was installed on almost all construction projects. In 1966 neoprene was used exclusively for transverse joints. While considerable development may yet be required to obtain optimum dimensional shapes and material requirements for this type of joint seal, to date it has considerably out-performed liquid-type joint sealers of the hot-poured or cold-applied type. If this type of joint seal continues to perform as it has in its first few years of application, it would appear that a considerable reduction in the previously discussed joint problems could be expected.

In 1960 joint spalling on an urban expressway in the western side of the State was called to our attention by maintenance personnel. The three most severely spalled joints were exposed and examined prior to repair to determine the cause of the deterioration. The first joint evidenced extensive surface spalling extending downward into the pavement no more than 2 in. at the joint face, and back from the joint face about 15 in. where the spall depth was about 1/2 in. Since the spalling was not deep, the transfer system was not exposed and its influence on this spalling could not be determined. It was noted, however, that near the surface the spalled concrete was chiefly mortar with very little coarse aggregate. In addition, over a considerable area, the plane-of-cleavage between sound pavement and the spall showed no evidence of broken aggregate, but rather of bond failures between coarse aggregate below and the mortar in the upper surface. Generally this indicates that the mortar is weak, because bond strength between mortar and coarse aggregate is proportional to mortar strength.

The second joint was a construction joint. Where the concrete was spalled most severely down to the levels of the dowels it was noted that all four dowels exposed tilted up 1/4 to 1/2 in. This misalignment is sufficient to the cause concrete-to-dowel binding and appears to have caused the spalling at this joint.

The third joint was another construction joint where, at the south side of the joint, the pour ended against month-old concrete on the north side (Fig. 75). The slab reinforcement was found to be only 7/8 to 1 in. from the surface on the spalled side of the joint. The joint was not moving properly, but was opening 6 in. further south where a crack had opened sufficiently to rupture the steel. The fact that the movement took place 6 in. south of the joint meant that only a few inches of the dowel extended across one side of the opening, resulting in absence of proper load transfer. Undoubtedly this condition was partially responsible for the pavement break-

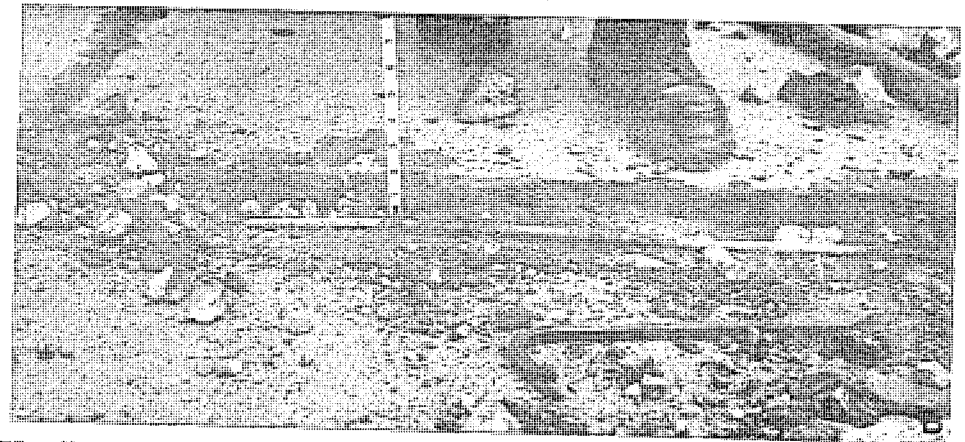
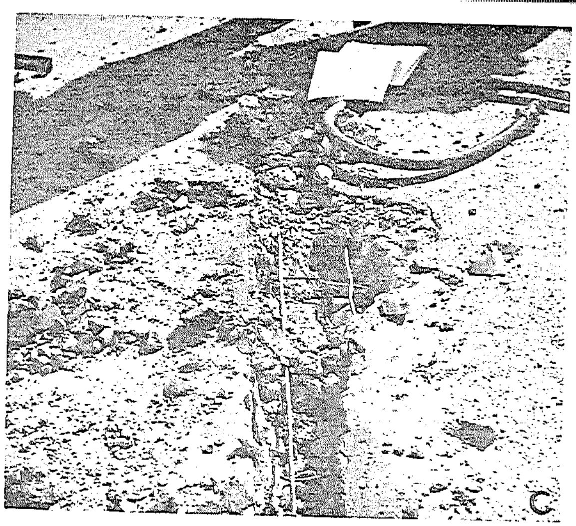
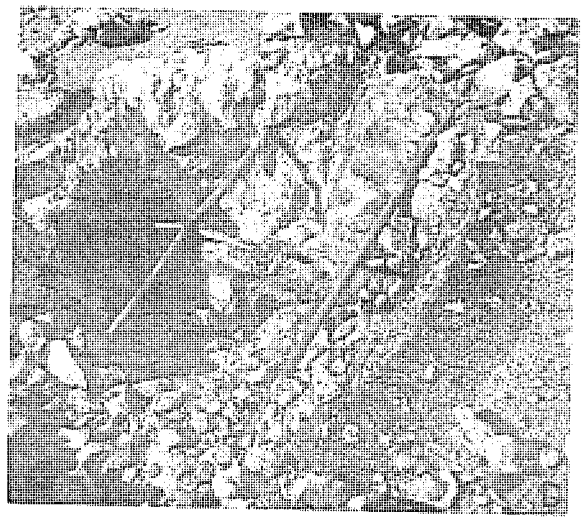
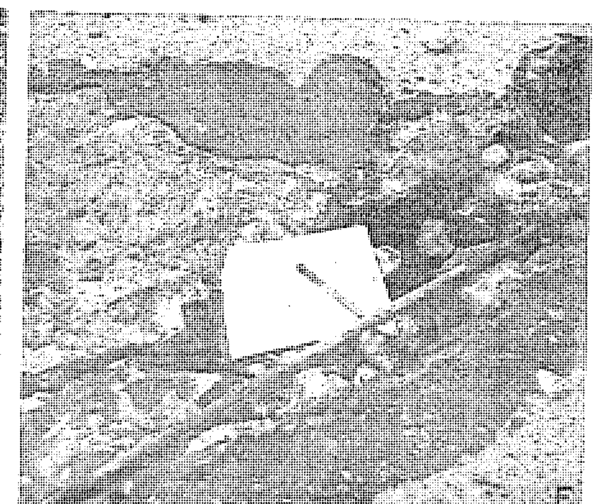


Figure 75. "Joint 3" where opening had occurred about 6 in. on the spalled side of the joint groove, is shown in an overall view (A). In the spalled area, reinforcement was embedded only 7/8- to 1 in. beneath the concrete surface (B). The reinforcing mat here was found to have been placed crosswise at the time of construction (C). A No. 4 gage wire was found to have broken within the slab, probably soon after construction (D, E), indicating that the pavement was opening at this point rather than at the joint.







age at this point. In addition, after more of the reinforcing steel was exposed in the slab south of the joint it could be seen that the mat was not correctly oriented, being crosswise with the transverse and longitudinal axis of the mat and the pavement opposed. The No. 00-gage wires at 6-in. spacing were oriented transversely, giving 0.688 sq in. to the lin ft, the No. 4-gage wires at 12-in. spacing were oriented longitudinally, giving only 0.159 sq in. to the lin ft, or less than a fourth of the proper steel area. This incorrect orientation of the steel mat undoubtedly caused the opening of the crack south of the joint and the early failure of the steel at this point. The load transfer system, however, must have caused considerable binding and freezing at the joint, and resulted in the slab movement taking place 6 in. away from the joint.

In 1962 a construction project on M 37 relocation was studied as a result of extensive transverse cracking noted in October. The pavement was poured between July 9 and August 11 of the same year. For the 20 days of concrete placement in the 9.95 miles of pavement the average transverse cracks per pour varied in number from zero to a maximum of 10.5 per 1,000 ft of pavement. However, some of this cracking was concentrated in local areas so that some 99-ft slabs had 3, 4, 5, and even 6 cracks per slab. The following analysis of various factors which may have contributed to this transverse cracking was made: 1) Temperature variation during paving operations and the following 24 hours was investigated for each of the 20 pours. This included variations between maximum and minimum temperatures. 2) The early strength of the concrete as reflected in compressive strength of cores and modulus of rupture of beams was studied. 3) Grading operations in relation to eventual paving failure, areas of cut, fill, and muck were established and transverse cracking was compared without significant difference in cracking for these three subgrade conditions. 4) Located cracks within a particular slab, and 5) Relative position or sequence of placement of the slabs that later showed most cracking within a particular pour was noted.

In the first four types of analysis no explanation for the unusual amount of cracking was found; however, the location of the cracking in relation to the particular pour did show significant results. The badly cracked slabs in almost all cases were those constructed during the first part of the day's operations. Since the cores did not show any significant difference in strength for the cracked sections, it appears that the concrete mix was satisfactory, and that conditions encountered during the setting period must have been responsible for the subsequent excessive shrinkage leading to transverse cracking. Climatic conditions contributing to rapid evaporation and consequent plastic shrinkage cracking include low humidity, heat from

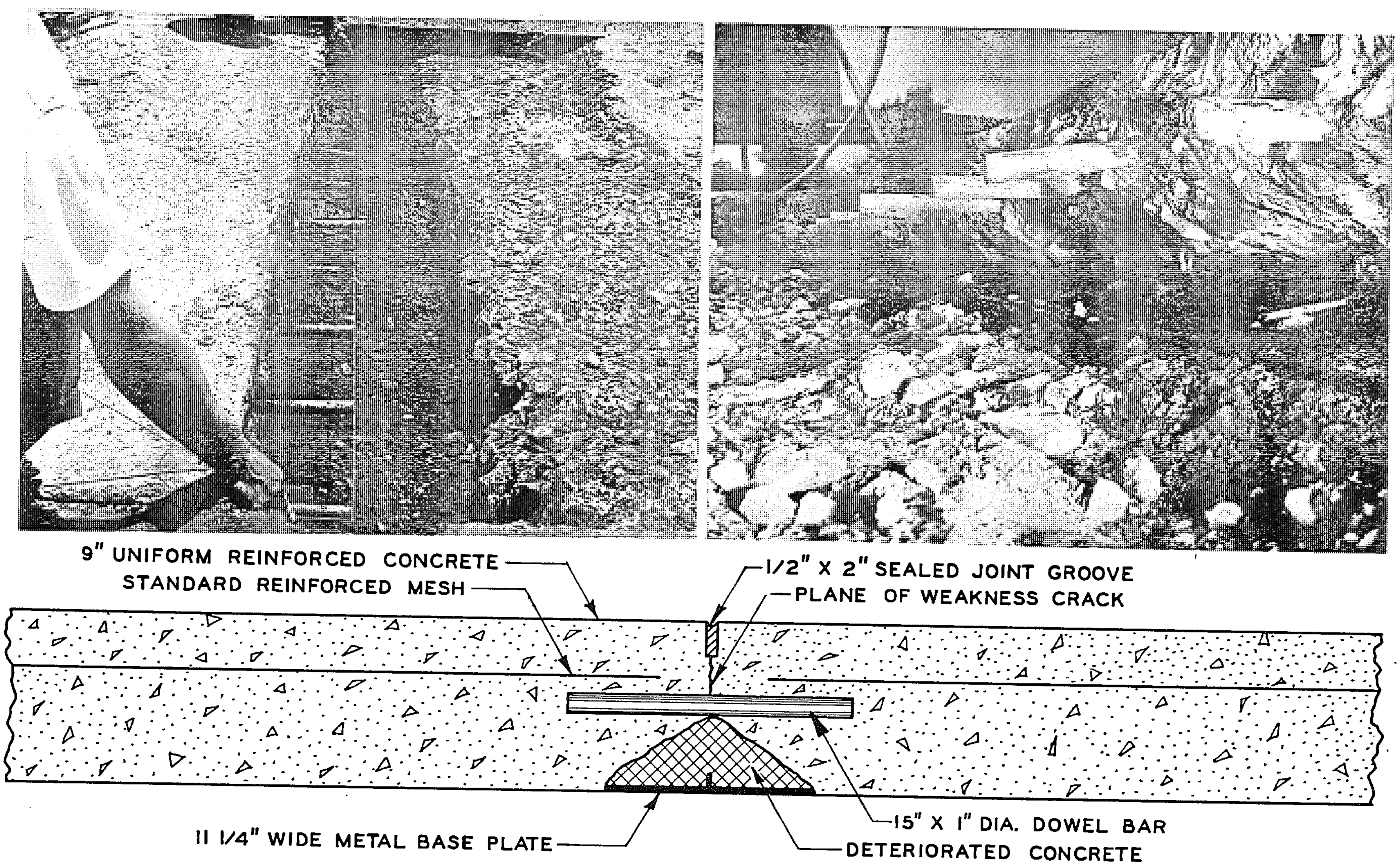


Figure 76. Pavement joint blow-up, dowel and joint groove misalignment observed during repair (upper left), concrete deterioration found below dowel bars (upper right) and schematic cross-section of deterioration.

continuous sunshine, and wind. Prompt application of the curing treatment to the freshly placed concrete is particularly critical on sunny and windy days and days with low humidity. Construction records were not sufficiently detailed to establish beyond a doubt that delay in placing the membrane curing compound, in combination with climatic conditions which would cause rapid evaporation of moisture from the concrete, led to the excessive shrinkage and early cracking of this pavement in certain locations. This is, however, the most likely cause for this cracking. Previous records indicate that cracking caused by improper curing of morning-poured slabs has often been experienced on numerous older projects.

In 1965 an investigation was made of the joint failures on a project constructed in 1953 on I 94. A total of 10 previous blowups had occurred and two additional ones occurred the day prior to observation and one on the day of observation. Observations on one of these joints during repair indicated that the dowel bars were out of alignment both laterally and vertically, and that the joint groove was not constructed symmetrically over the base plate parting strip (Fig. 76). Concrete below the dowel bars was saturated with moisture and completely deteriorated. The cause of blowups on this project were ascribed as follows:

"Blowups and other evidence of poor joint performance on this project may result from one or more of several causes. Over several years, dirt has infiltrated progressively into the joints, preventing joint closure during pavement expansion cycles. In addition to this normal infiltration, water and chloride solution resulting from ice and snow removal has seeped into the joints and could have been trapped by the base plate. Alternate freezing and thawing, coupled with the detrimental effect of chloride solution on concrete, may have accelerated deterioration of the concrete below the dowel bars. As a result, compressive forces--caused by restraint to concrete expansion resulting from moisture and temperature--are greatly increased, at the same time the concrete area resisting these forces is decreased about 60 percent." This particular project not only showed poor performance with respect to blowups but was also one of the worst with respect to transverse cracks.

In September 1965, two adjacent construction projects on US 127 were investigated intensively. The first project was constructed in 1947 and the second in 1949. Almost every joint in the 1947 project showed distress to some degree, with about 50 percent of the joints in very bad condition. A typical joint is shown in Figure 77. The concrete at the joint had disintegrated in a wedge shape, tapering up to the dowel bars from the plastic base plate. The concrete above the dowels was fractured into small pieces, as

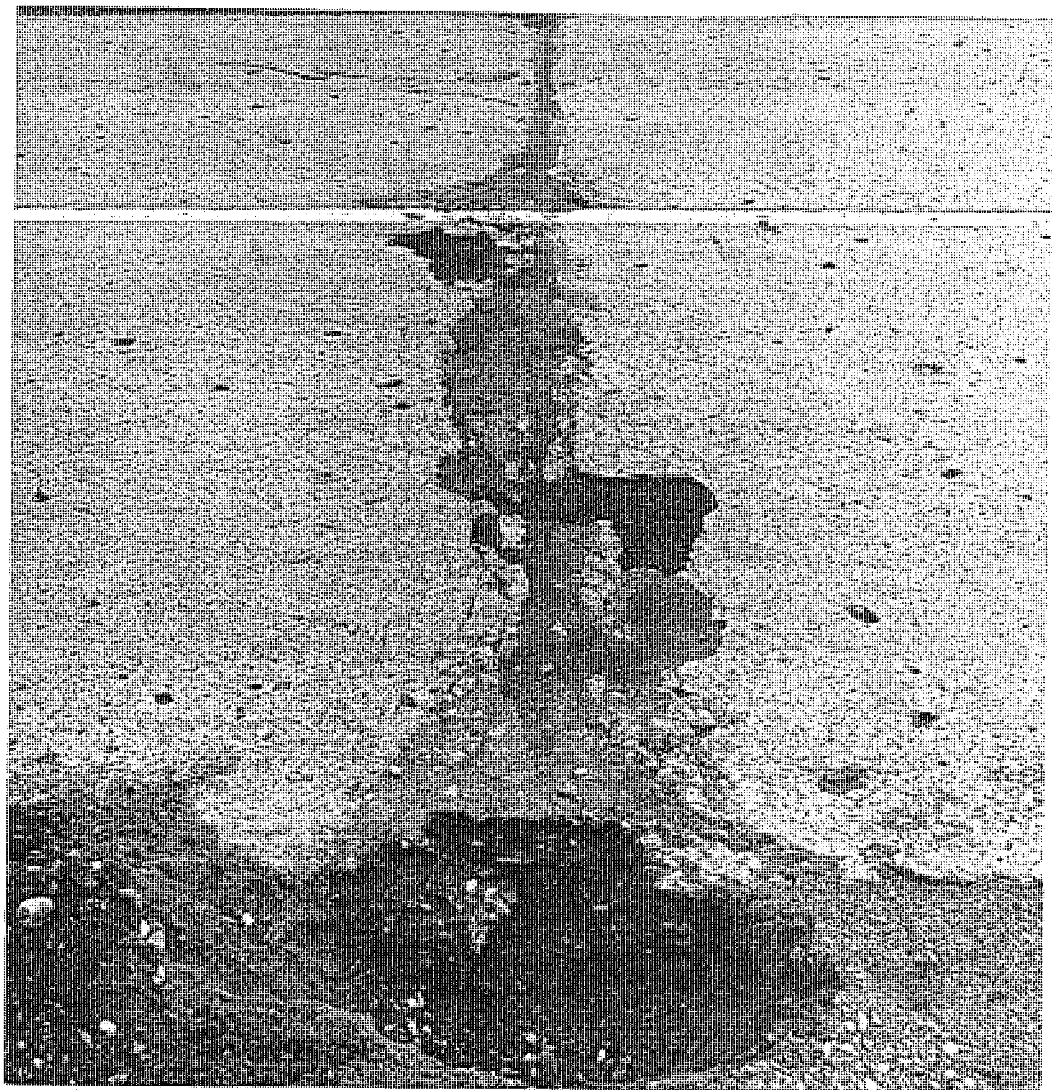


Figure 77. Distressed joint with end exposed.

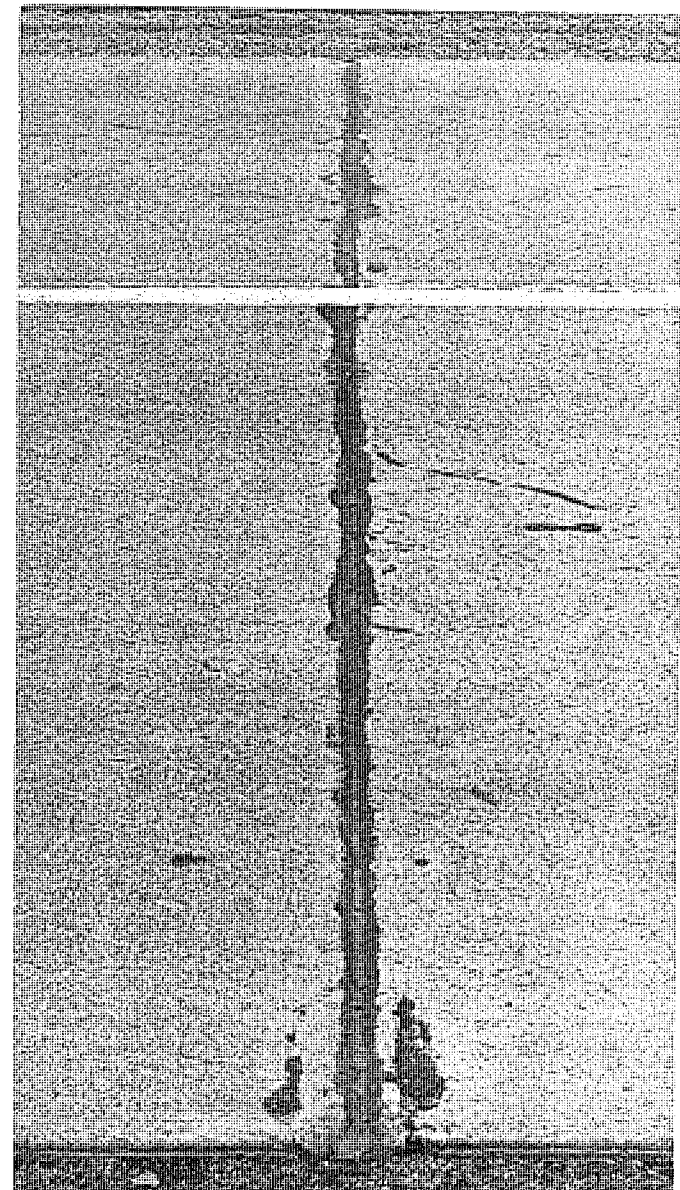


Figure 78. Typical joint in good performance area.

though subjected to severe compression. The joints on the second project, constructed in 1949, were in excellent condition for approximately 3.1 miles with almost no distress (Fig. 78). On the remaining 1.6 miles of this project the joints had serious spalls where maintenance was required. A study of the design and materials factors as well as a study of the subbase material indicated that the only apparent difference in the performance of the pavement joints could be associated with the sources of coarse aggregate used in the concrete. The area of the second project where the performance was excellent after 16 years of service had a limestone coarse aggregate in the concrete. As shown earlier, however, this good performance on a larger statistical sample correlates with the generally lower amount of soft, non-durable material found in limestone sources.

In 1966 a reappraisal of transverse joint baseplates was made. The baseplate used in Michigan construction practices since 1946 had two purposes: 1) It was intended to prevent infiltration of fine inert particles from base course and shoulder construction into the joint opening, and 2) the base plate was to serve as a support for the dowel assembly in place of 6- by 6-in. sand plates or 2-in. wide continuous bearing plates attached to wire supports of the dowel-bar assembly. The base plate also furnished support for a 1-in. high parting strip placed directly under the surface groove to control the cracking at the joint.

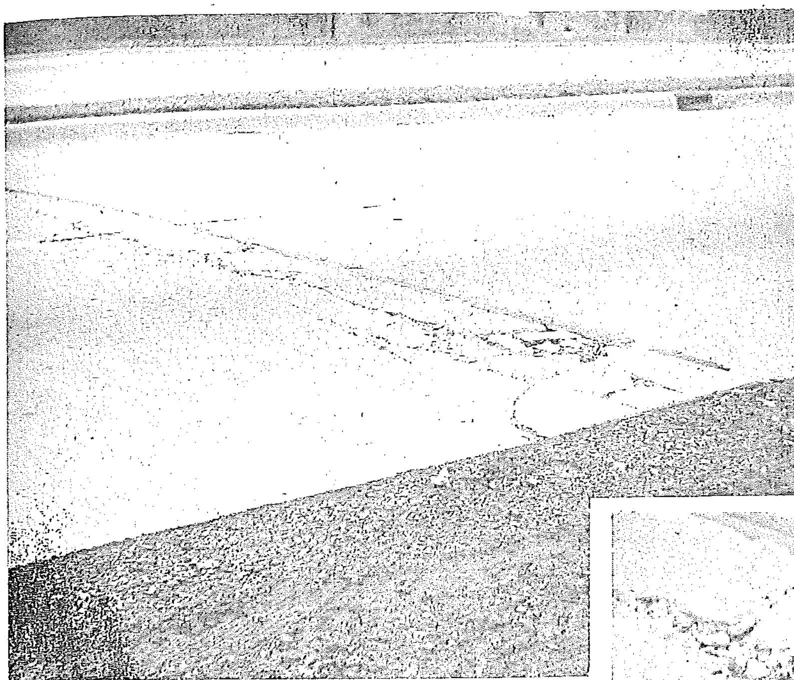
Over the years since 1946 several changes were made in the design and material of baseplates. In the 1950's rubber and plastic were approved as substitutes for galvanized steel but for various reasons were discarded as being unsatisfactory. Also, during this period, changes in base course construction were made to upgrade the physical characteristics of the subbase. Current specifications require two layers of granular material for a total of 15 inches. A porous material is permitted for the bottom 11 in. followed by a selected subbase on the top 4 in. which was designed to prevent loss of density due to drying and rutting and to provide a stable surface for paving forms and to prevent infiltration of fine uniform sands into the pavement joint.

In connection with field investigations of pavement joint problems many joints had been examined during repair. Many of the contraction joints showed various degrees of spalling and concrete deterioration at the bottom of the joint. It was quite obvious that failure of the joint seal allowed the joint space to fill with soil materials. In addition, water and maintenance chemicals had also entered the joint and resulted in a triangular zone of concrete deterioration with the triangle's base about the width of the baseplate and it's apex centered at the joint at the height of the dowel bars. It appeared that the baseplate was trapping water and maintenance chemicals in this triangular zone, resulting in a much reduced joint face area which could result in later blowup problems when compressive forces from pavement expansion due to moisture and temperature changes were exerted on this reduced effective area. As a result the Department decided to eliminate baseplates from pavement construction.

In 1957, three damaged joints on I 94 were called to our attention by county maintenance personnel. This pavement was six years old at the time of the examination. The reason for the failures of the first two joints shown in Figures 79 and 80 was the misplaced joint groove, approximately 6 in. too far west. This resulted in reinforcement extending through the joint and dowel bars embedded 13-1/2 in. into one slab and only 1-1/2 in. or less into the adjacent slab. The reason for the concrete spalling at the corner of the third joint was no doubt due to the twisting and misplaced reinforcement at this point which resulted in only 3/4 in. of concrete cover (Fig. 81).

In 1954 unusual cracking at joints was investigated on a project on US 2 in the Upper Peninsula. This pavement was placed between July 16 and 30, 1954. It was first noted by excavating the shoulder alongside the pavement joint that the baseplate assembly was not in the proper location with respect to the joint groove in the pavement. A survey of 17 contraction joints was made on this project by means of a Research Laboratory designed electronic instrument which would indicate the position of steel embedded in the concrete pavement. As shown in Figure 82, it was determined that the sealed joint groove had been skewed to the location of the dowel bar assembly from approximately 3 to 10 in., thus the formed plane-of-weakness in some cases completely missed the dowel bars at one end of the pavement, and in other cases only a small portion of the dowel bar extended into the adjacent slab.

It should be noted that all of the performance problems previously discussed under Objective V were caused by inadequate inspection and control of load transfer assemblies or pavement reinforcement. The design of load transfer assemblies has improved since this early postwar period and thus the frequency of some of these problems should also be reduced in later pavements. However, closer inspection of the proper placement of load transfer assemblies and alignment of the dowel bars, together with correct positioning of reinforcing steel in relation to transverse joints and to depth within the slab, would remedy the poor performance problems illustrated here.



Condition of pavement joint on October 30, 1957.

Broken concrete removed to level of reinforcing steel. Reinforcing steel was twisted and tilted upward at joint.

Detailed view at pavement edge showing reinforcing steel passing through joint and dowel extending 1-1/2 inches across joint.

Detailed view showing two edge dowels exposed. Load transfer assembly was 6 inches too far east of pavement joint.

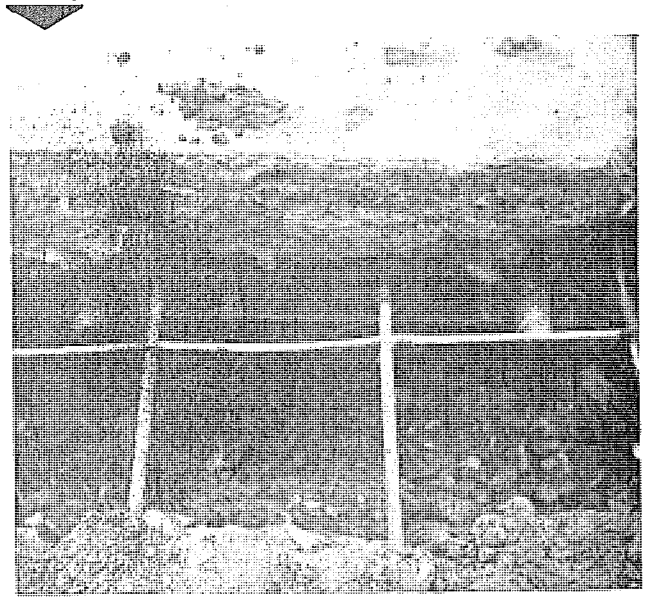
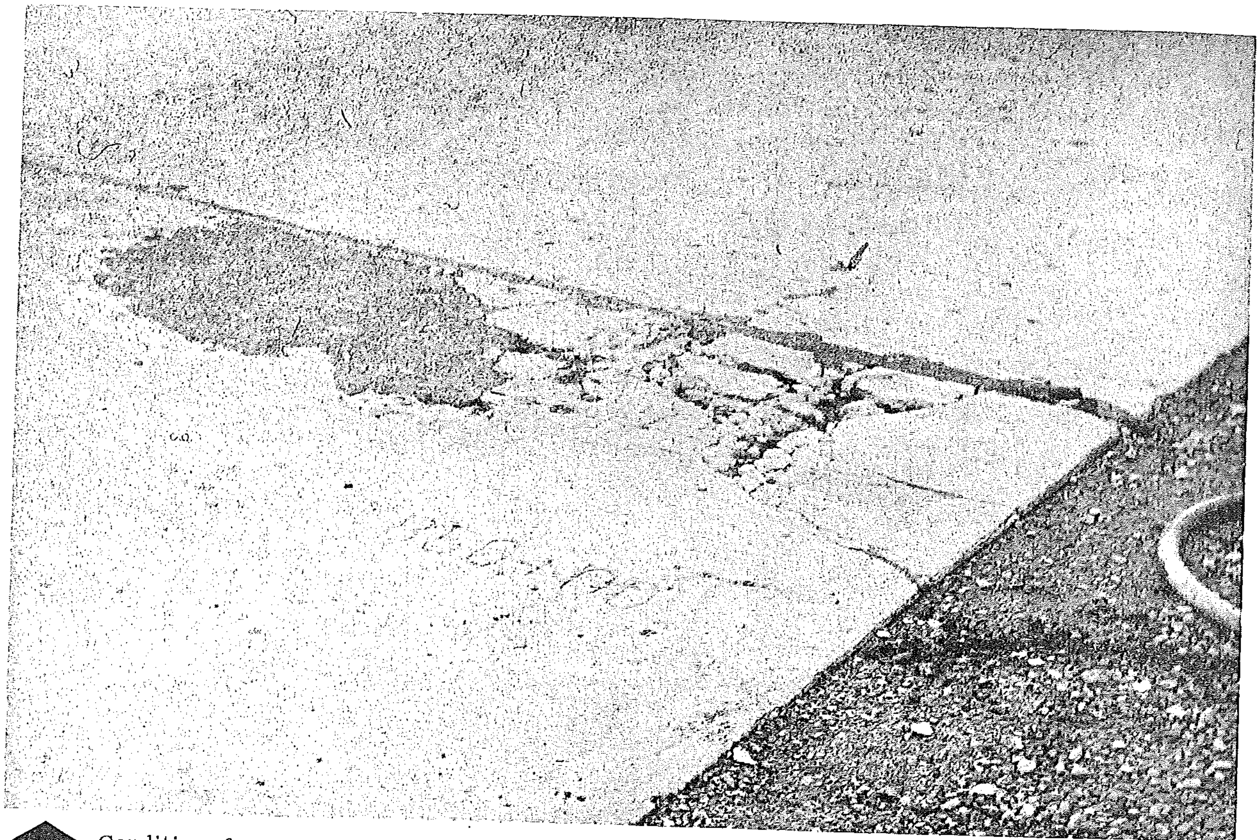
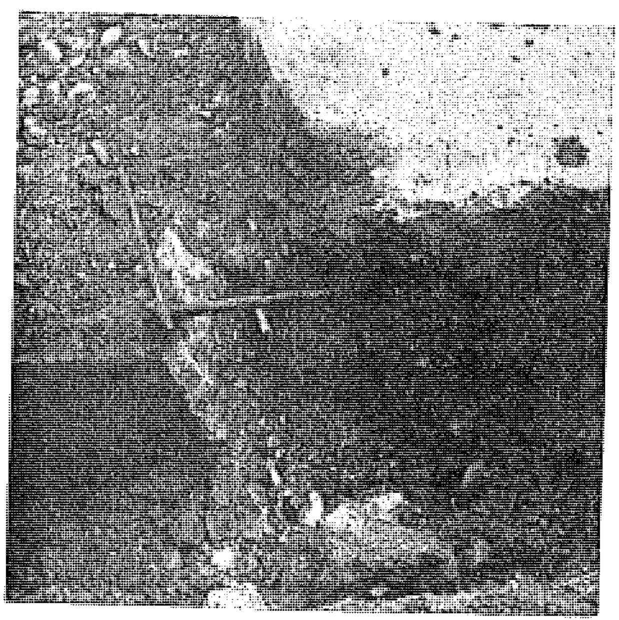


Figure 79. Condition and cause of joint problems.



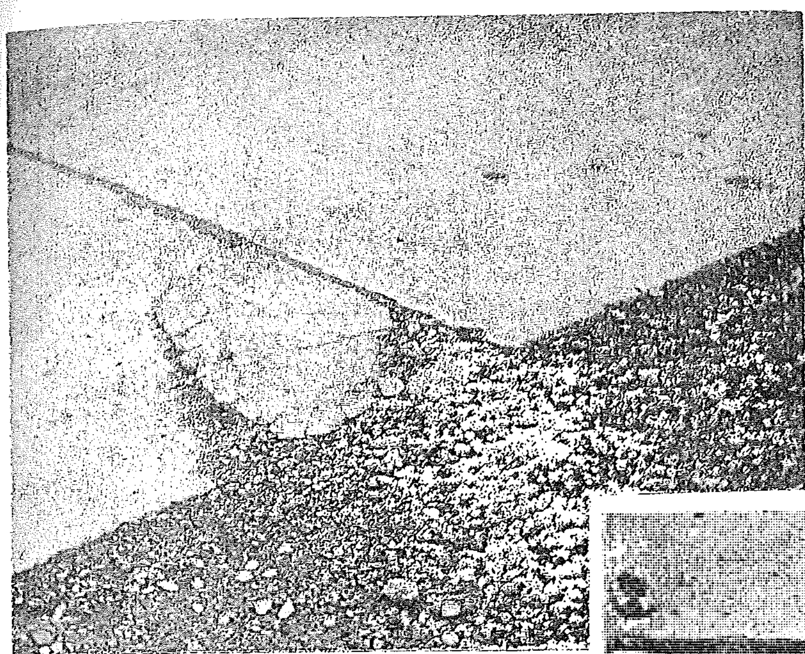
Condition of pavement joint on October 30, 1957.

Two edge dowels exposed showing improper location of load transfer assembly.



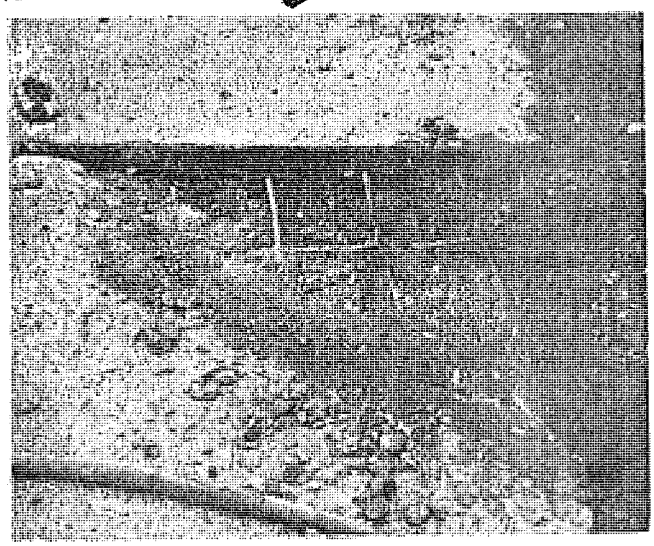
Edge dowel exposed showing 1-1/2 inches of lap across joint. Reinforcing steel passes through joint and although "necked down" was not broken.

Figure 80. Condition and cause of joint problems.



Condition of pavement joint, October 30, 1957.

Broken concrete removed to level of reinforcing steel.



Reinforcing steel shown to be within 3/4 inch of pavement surface at transverse joint.

Edge dowel exposed showing 5-1/2 inch rather than 7-1/2 inch extension into adjacent slab.

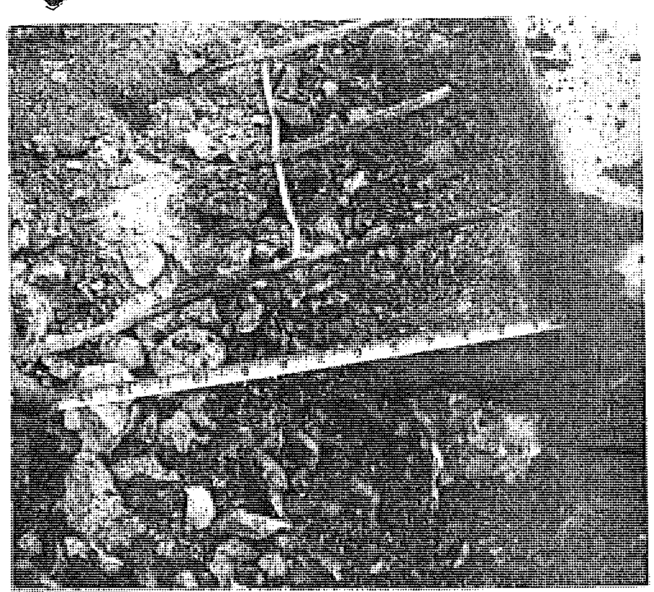


Figure 81. Condition and cause of joint problems.

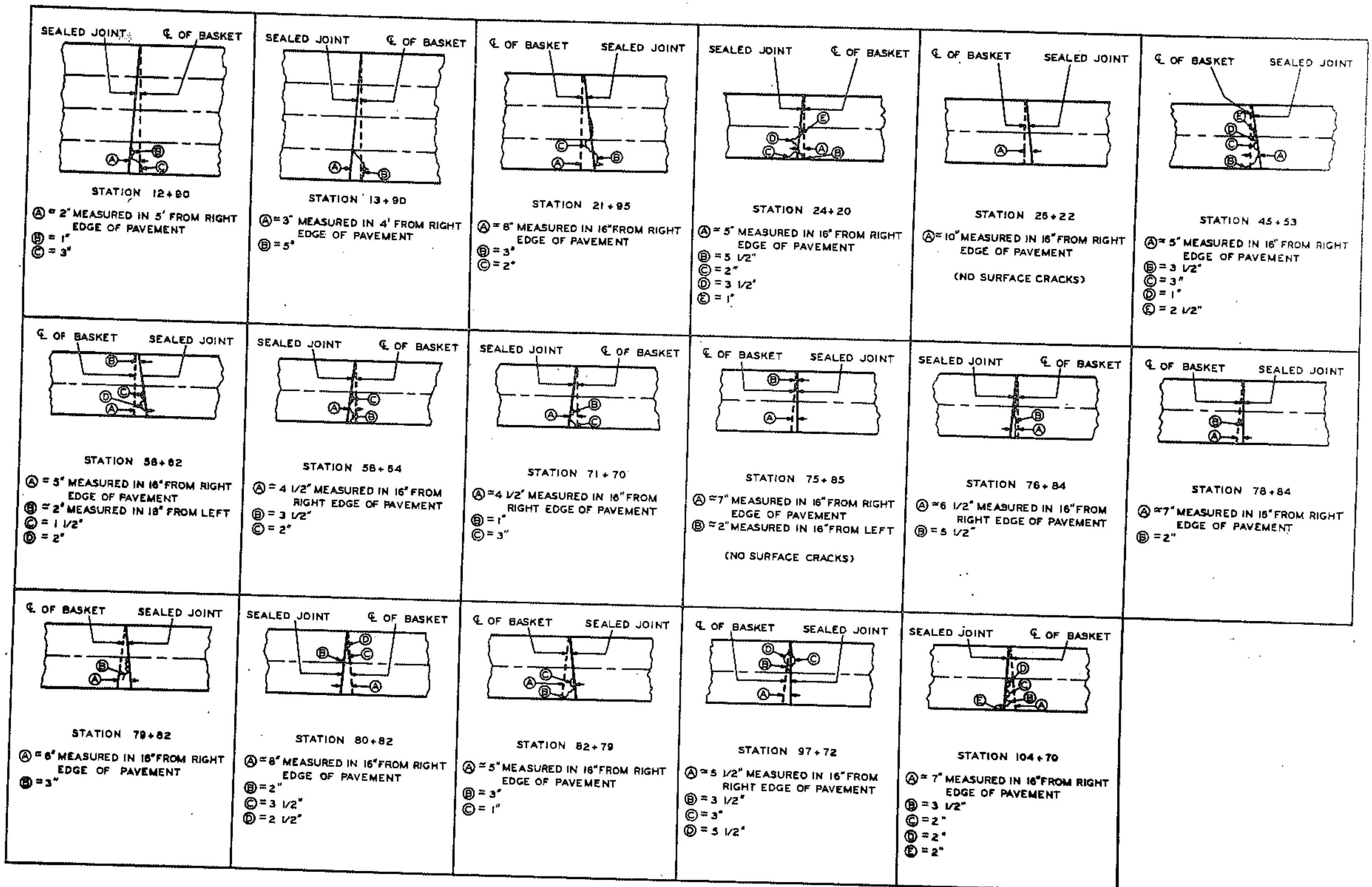


Figure 82. Condition and cause of cracking at various joints on project on US 2.

Special Studies

As a part of this research study, Michigan took part in the Purdue University National Cooperative Highway Research Project to compare different methods of measuring pavement condition. This study, conducted in 1963, involved 45 pavement sections of three types (rigid, flexible, and overlay). These sections were rated by a lay panel, the AASHO Road Test panel, and the Highway Research Board Committee on Pavement Condition Evaluation. The extent of cracking and patching was determined for each section. Roughness, and profilometer measurements were made using roughometers of eight different types, the BPR-type roughometer, the AASHO Slope Profilometer, CHLOE Profilometer, Kentucky acceleration device, Texas Texture Meter, University of Michigan Truck Mounted Profilometer, General Motors Corporation Rapid Travel Profilometer and the Purdue University tire pressure instrument.

Michigan's roughometer, a BPR type, was used in this study. This instrument has two means of measuring roughness. The first or conventional means which involves the use of a mechanical integrator consists of a cable-driven drum with a clutch arrangement that permits drum movement to be measured in one direction only. Values from the integrator are expressed in terms of inches per mile. The second method involves a five-channel limit-set indicator which records impulses from a 2g accelerometer mounted on the roughometer frame. Values from the accelerometer are expressed in g's per mile (g = unit of force equal to the force of gravity). Limit switches for separating g levels were adjusted for five different conditions depending on the roughness of the pavement.

The correlation study indicated that the coefficients to be used for the AASHO Model Equation for the Michigan roughometer were as follows:

For rigid pavements:

$$\text{Present Serviceability Index} = 5.39 - 0.0076\ F - 0.006\ \sqrt{C + P}$$

of

$$\text{PSI} = 5.72 - 0.0018\ G - 0.006\ \sqrt{C + P}$$

where:

F = roughness in inches per mile (mechanical integrator)

G = roughness in g's per mile (acceleration measurement)

C = major cracking in ft per 1,000 sq ft of area

P = bituminous patching in sq ft per 1,000 sq ft of area

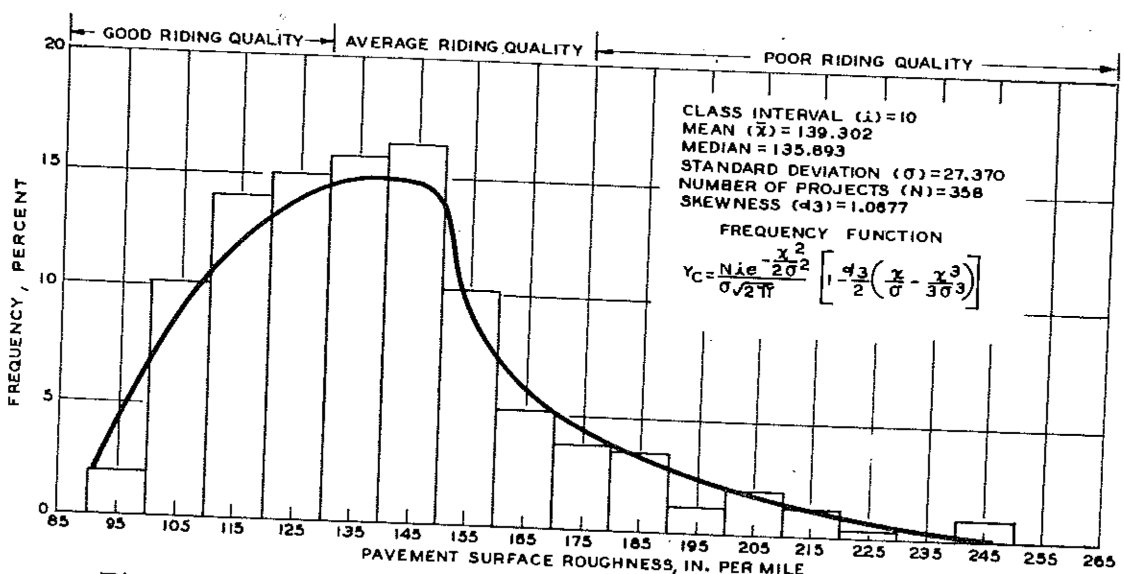


Figure 83. Frequency distribution of initial surface roughness values from rigid pavements constructed in Michigan; 1951-62.

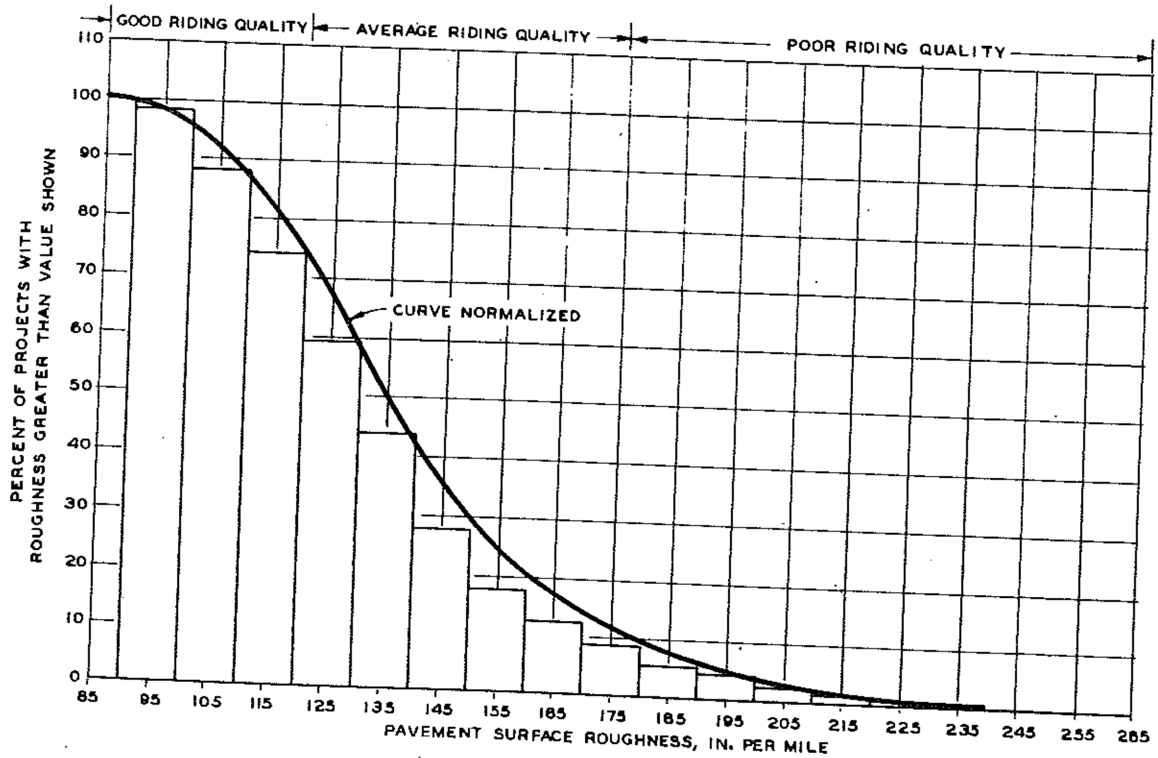


Figure 84. Cumulative frequency distribution of initial surface roughness values from rigid pavements constructed in Michigan; 1951-62.

In this correlation study the Michigan roughometer (mechanical integrator) had the highest correlation with the AASHO model equations and the smallest standard error of estimate of all roughness measuring instruments when used on rigid pavement.

Pertinent findings (13) which have a bearing on this study are as follows:

1. The lay rating panel, on the average, rated pavements higher than the professional people.

2. Serviceability equations using the AASHO mathematical model were developed for each piece of equipment.

3. Equations were developed that permit prediction of serviceability using only equipment measurements. These equations showed, in general, very slightly lower correlation coefficients and higher errors of estimate than the AASHO model equations.

4. The field test results indicated that from the standpoint of precision in predicting serviceability little difference existed in roughness measuring equipment.

Another special study on initial roughness and serviceability indices was conducted in cooperation with the Division of Highways, Department of Public Works and Buildings of the State of Illinois. This study was proposed in a letter from W. E. Chastain, Sr., Assistant Engineer of Research and Planning of Illinois to John C. Mackie, Highway Commissioner of Michigan. With the approval of Howard E. Hill, Managing Director and under this HPR project, the initial roughness and the initial Present Serviceability for all newly constructed rigid pavements from 1951 through 1962 were computed (358 construction projects). The objective of this study as proposed by Chastain was to compare the riding quality that is being obtained by various states since Indiana, Iowa, Michigan, Missouri, and Illinois each had a BPR-type roughometer and measurements are mutually correlatable because of prior correlations with AASHO Road Test profilometer. Some of the data proposed for this study have interest within the objectives of this project and are therefore reported here.

Figure 83 shows the frequency distribution and Figure 84 the cumulative frequency distribution of initial roughness values for these 358 pavement projects. The frequency distribution is quite skewed, with a majority of projects in the good or upper average categories. The mean unweighted

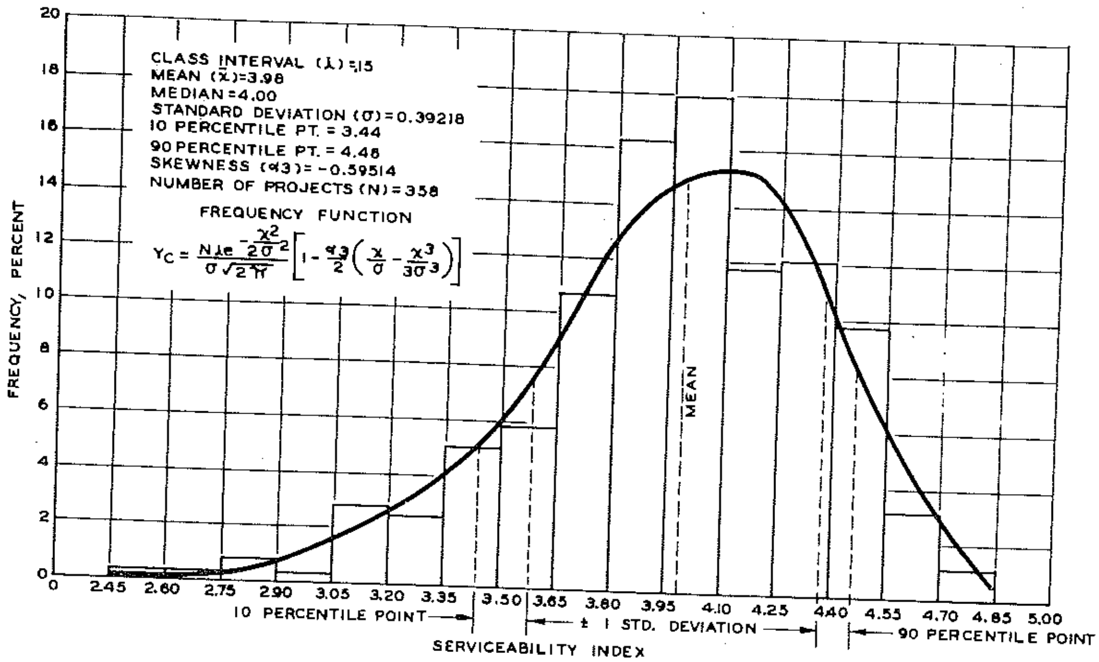


Figure 85. Frequency distribution of initial Serviceability Index values for new rigid pavement constructed in Michigan; 1951-62.

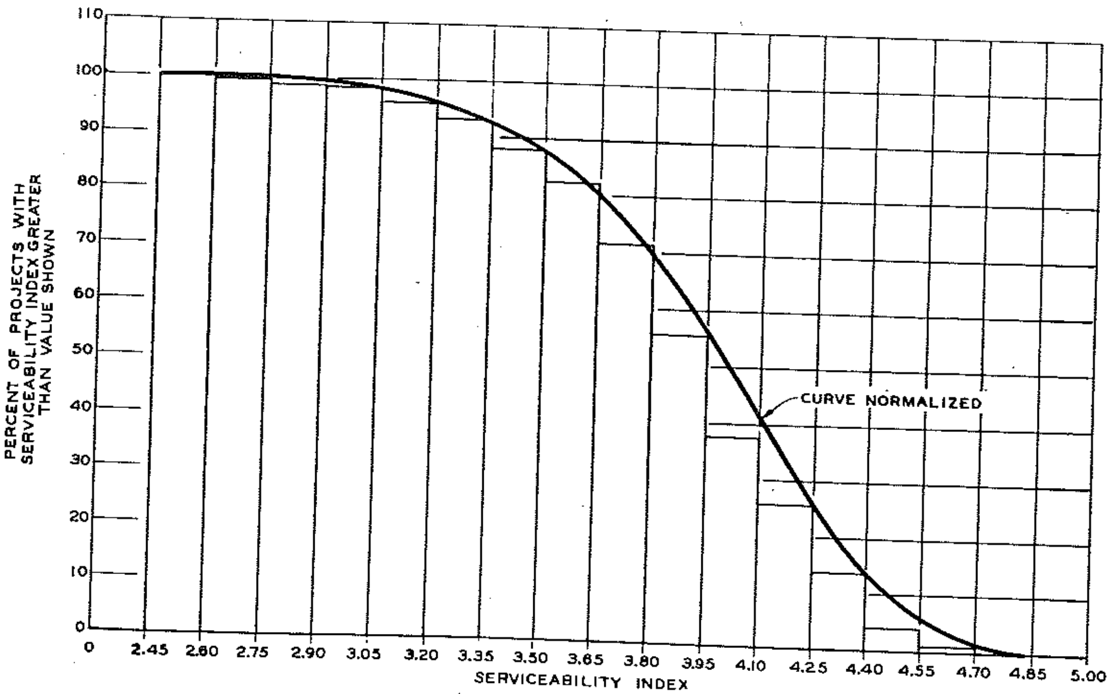


Figure 86. Cumulative frequency distribution of initial Serviceability Index values for new rigid pavement constructed in Michigan; 1951-62.

initial roughness is approximately 139 inches per mile and the median approximately 136 inches per mile. "Good," "average," and "poor" categories as shown in Figures 83 and 84 for riding quality of newly constructed pavement have been established arbitrarily on the basis of roughness distributions in Michigan studies and are shown in relation to the roughness distribution.

In Figures 85 and 86 the frequency distribution and the cumulative frequency distribution of the initial serviceability indices for the same projects are shown. The median index value is approximately 4.0 but the entire distribution encompasses values from 4.85 to 2.45. With these data normalized it can be anticipated that 10 percent of the projects will have an initial PSI of 4.46 or higher, 67 percent of the projects will be between 3.59 and 4.38, and 10 percent of the projects will have a PSI of 3.44 or below.

Performance Ranking Study

One reason for taking condition surveys or roughness measurements is their value not only in measuring current performance but also predicting future performance. As previously discussed under roughness, the wide variation in initial roughness makes predicting future performance based on roughness measurements of little practical value. Between the ten- and fifteen-year service period the increase in roughness measurement will have slight correlation with structural deterioration but by this time the pavement distress may be obvious by casual inspection. It would be of interest, however, if at any earlier period, say after five years of service, the performance at fifteen years could be predicted.

In Table 3 two methods of predicting future performance are indicated. For the first case the statistical correlation coefficient for five-year surveys is compared to fifteen-year surveys for the same condition survey variable. For the second case the correlation between dissimilar condition survey variables is shown. The intercorrelation between external corner breaks and internal corner breaks and between internal and external spalls with blowups are particularly significant. Other survey variables at five years correlate to a less significant degree with other survey variables at fifteen years, but in general the predicting power to fifteen years for five-year surveys is quite good.

Another method of indicating the ability to "predict" performance from condition surveys is shown in Figure 87 where the project performance at the end of five years is divided into five groups and then the individual group performance is observed at the end of ten and fifteen years of service. A statistical comparison indicates the same thing: The average rank correlation is 0.82 which is highly significant thereby substantiating the predicting power of condition survey variables in estimating future deterioration.

TABLE 3
INTERCORRELATIONS OF VARIOUS
CONDITION SURVEY VARIABLES BETWEEN SURVEY PERIODS

5-Year Survey Variable	15-Year Survey Variable	Correlation Coefficient (r)
% External Corner Breaks	% External Corner Breaks	0.89
Longitudinal Cracking ft. per mile	Longitudinal Cracking ft. per mile	0.83
Transverse Cracks per slab	Transverse Cracks per slab	0.61
% Internal Corner Breaks	% Internal Corner Breaks	0.49
% External Corner Breaks	% Internal Corner Breaks	0.87
% Internal Spalls	Blowups per 100 Joints	0.81
% External Spalls	Blowups per 100 Joints	0.72
Spalls in Slab Surface/Mile	% External Corner Breaks	0.72
% Internal Corner Breaks	Blowups per 100 Joints	0.63
Spalls in Slab Surface/Mile	% Internal Corner Breaks	0.59
Infiltration Cracks/Lane Joint	% External Spalls	0.55
Infiltration Cracks/Lane Joint	Corner Breaks at Cracks/Mile	0.51
Spalls Along Centerline/Mile	Corner Breaks at Cracks/Mile	0.47
Spalls at Cracks/Mile	Corner Breaks at Cracks/Mile	0.46
Transverse Cracks/Slab	Blowups per 100 Joints	0.43
Transverse Cracks/Slab	Deterioration in sq. ft./mile	0.42

Note: Correlation Coefficient of 0.84 - highly significant, 0.45 - significant.

Methods of Performance Evaluation

As a result of the AASHO Road Test the Present Serviceability Index has been used rather extensively by state highway departments as a means of evaluating pavements. The largest and overriding factor in this index is pavement roughness and while certain condition survey factors are incorporated into the index, their effect on the index is almost negligible. This study has indicated that if structural performance of the pavement is to be evaluated, the Present Serviceability Index performs poorly. The reason for this is that initial roughness plays such a predominant part in the Index that initial PSI values, as shown previously in Figure 86, range from 2.45 to 4.85. With this range in PSI values for newly constructed pavements it is apparent that the use of this Index to measure the performance of pavements after years of service is inappropriate. It can be readily shown that pavements with ten, fifteen, or twenty years of service exhibiting numerous cracks, spalls, corner breaks, etc. will still result in a higher PSI value than new construction which has been built with poor riding quality. Also, the early signs of pavement distress such as transverse cracking and spalling are not reflected in the PSI measurement. As an extreme illustration of this we note that the tail of the distribution of PSI extends to 2.45 for new pavements. However, a study of Michigan Pavements recommended for resurfacing or replacing were measured and it was found that their average PSI was approximately 2.5. The use of condition survey variables to develop a subjective rating model (Performance Rating Factor) or an objective rating model (Structural Deterioration Index) does not have this limitation. In addition it has been shown that both of these condition survey indices are better predictors of future values than PSI.

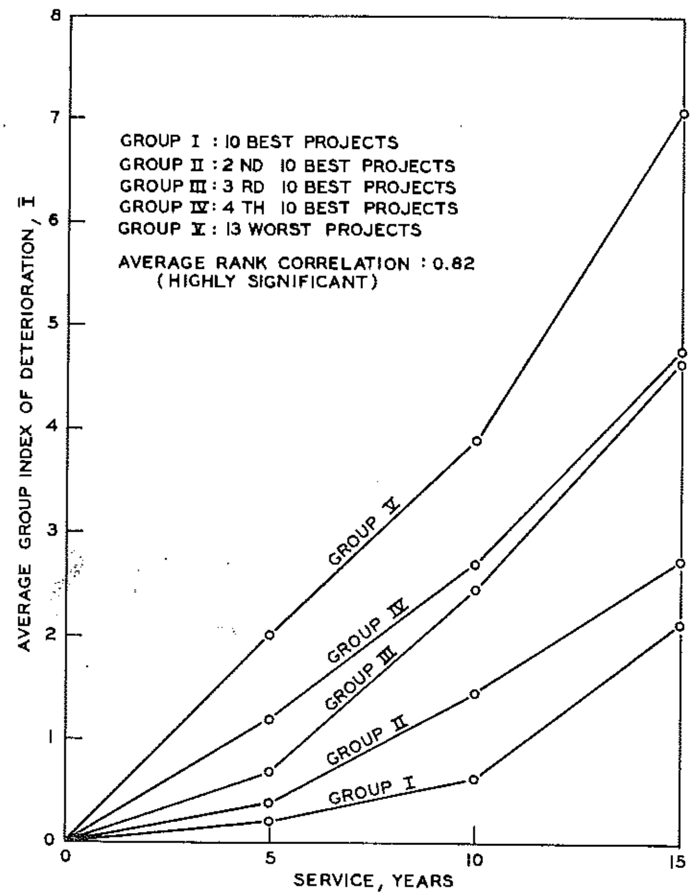


Figure 87. Five-year structural deterioration index as a predictor of future performance.

CONCLUSIONS

The following general conclusions may be drawn from the performance evaluation of postwar concrete pavements (1,880 miles) in Michigan:

1. Condition surveys indicate an extreme variability in performance between projects with respect to transverse or longitudinal cracking, corner breaks, spalls, blowups, mud-jacking, and patching. For most all of these types of deterioration the bulk of the construction projects performed rather well with no serious distress after ten or fifteen years. However, a small number of projects exhibited extremely poor performance in some of these types of deterioration.

2. Since most projects performed satisfactorily, and the same basic design was used for all projects, it is apparent that causal factors for the poor performance of a few projects are much more likely to be related to materials, construction factors, or the environmental factors of climate and traffic loading.

3. Statistical and graphical analysis indicated a significant correlation between traffic and coarse aggregate quality (as measured by Heterogeneity Index or soft, non-durable content) with pavement structural performance.

4. All condition survey indicators of performance (transverse cracking, longitudinal cracking, external or internal corner spalls, deterioration, and patching) showed that traffic had a marked effect on traffic - passing lane performance differences. General structural performance as measured by the Depreciation Index showed that the performance was 65 percent poorer for traffic lanes as compared to their associated passing lanes.

5. Performance indices (of either the subjective or objective type) based on condition surveys are much more valuable in indicating structural deterioration than the Present Serviceability Index based primarily on roughness. Moreover performance indices based on condition surveys serve to measure the "remaining useful life" of pavement while the Present Serviceability Index is nearly useless in this respect. The PSI fails to measure early signs of pavement distress and may be used to predict

"remaining useful life" only when pavement deterioration has reached a stage where prediction is no longer required.

6. An objective performance model based on commercial traffic and soft, non-durable content of the coarse aggregate was developed, which for service periods up to fifteen years explained 65 percent of the general structural performance variance. The remaining 35 percent of the variance remains unexplained and is due to local environmental and construction conditions, performance variables not available to us, and errors in estimating of traffic and soft, non-durable content.

7. Signs of short service life appear in the five-year condition surveys. These early signs are significantly correlated with later structural performance as measured at the ten- and fifteen-year surveys. Thus, after five years of service, it should be possible to determine which projects will fail prematurely.

8. Blowup frequency is considerably higher for pavements constructed with coarse aggregate containing greater amounts of soft, non-durables and it is assumed that blowups are causally related to this type of deleterious material.

RECOMMENDATIONS

1. Michigan's postwar pavements have generally performed quite well. Thus, it is recommended that no major changes in concrete design practice be made. This includes pavement thickness, joint spacing, and joint load transfer. Minor changes already made should improve pavement performance over that reported here. Particularly significant is the change to neoprene joint seals in place of hot-pour rubber-asphalt seals. Periodic winter surveys have indicated a lack of infiltration of foreign material into the joint groove for seals placed as early as 1962. It is expected that blowups and joint spalling should be markedly reduced as a result of this more recent design change.

2. On the basis of the early postwar pavements (1946-1954) where joint blowups and spalling are most prevalent after ten to fifteen years of service, the most feasible way of improving pavement would be a more restrictive specification on the soft, non-durable content of the coarse aggregate. From 1946 to 1954, the maximum allowable soft, non-durable content was 3.0 percent. In the 1963 specifications for 4A and 6A aggregate for concrete this maximum was reduced to 2.5 percent, while for 6AA aggregate, a premium aggregate for structures, the maximum is 2.0 percent. From this study an economic investigation appears warranted to determine if specifying a premium aggregate such as 6AA with a maximum soft, non-durable content of 2.0 percent should be required for pavements as well. This could be done by specifying aggregate for some projects throughout the state with a maximum of 2.0 percent soft, non-durable content and comparing bid prices to determine if this premium price would not be justified in line with the demonstrated reduction in pavement deterioration noted in this report. (One recent specification change, a reduction in the size of the coarse aggregate from 95 to 100 percent passing the 2 in. sieve to 95 to 100 percent passing the 1 in. sieve, should have a marked improvement in the surface performance of the concrete pavement. By reducing the size of the coarse aggregate, the size and seriousness of surface pop-outs from the permissible deleterious content of the coarse aggregate should be greatly decreased.)

3. Continue the present condition survey program of taking initial and five-year surveys on all projects, and taking ten- and fifteen-year surveys

when possible, since condition surveys are the only reliable way of gaging structural performance. Continue roughness surveys on new projects, and at five-year intervals thereafter when time permits, as a measure of pavement performance with respect to riding quality, with the realization that roughness or PSI is an unreliable indication of the pavements structural performance.

NOTE

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

REFERENCES

1. Hutchinson, B. G., "Principles of Subjective Rating Scale Construction," Highway Research Record No. 46, 1963.

2. "The AASHO Road Test," Highway Research Board Special Report 61A, Pub. 816, 1961.

3. Harman, Harry H., "Modern Factor Analysis," University of Chicago Press, 1960, pp. 1-23, 154-157, 337-348.

4. Walker, R. D., "Identification of Coarse Aggregates That Undergo Destructive Volume Changes When Frozen in Concrete," NCHRP Report No. 12, 1965.

5. Reagel, F. V., "Chert Unfit for Coarse Aggregate," Engineering News Record, Vol. 93, No. 9, 1924.

6. Legg, F. E., Jr., "Freeze-Thaw Durability of Aggregate in Concrete," Highway Research Board Bulletin No. 143, 1956.

7. Schuster, R. L., "A Study of Chert and Shale Gravel in Concrete," Highway Research Board Bulletin No. 305.

8. Swenson, E. G., Chaly, V., "Basis for Classifying Deleterious Characteristics of Concrete Aggregate Materials," Journal of the American Concrete Institute, Vol. 27, No. 9, May 1956.

9. Cantrill, C., Campbell, L., "Selection of Aggregate for Concrete Pavements Based on Serivce Records," ASTM Proceedings, Vol. 39.

10. Kellerman, W. F., "Effect of Type and Gradation of Coarse Aggregate Upon the Strength of Concrete," Public Roads, Vol. 10, No. 4, 1929.

11. Stanton, T. E., Jr., "Influence of Cement and Aggregate on Concrete Expansion," Engineering News Record, Vol. 124, No. 5, 1940.

12. White, L. V., Peyton, R. L., "Condition of Concrete Pavements in Kansas as Affected by Coarse Aggregate," Highway Research Board Proceedings, 1945.

13. Yoder, E. J. and Milhous, R. T., National Cooperative Highway Research Program Report 7, "Comparison of Different Methods of Measuring Pavement Condition," Interim Report, Highway Research Board, 1964.

14. Woods, K. B., Sweet, H. S., and Shelburn, T. E., "Pavement Blowups Correlated with Source of Coarse Aggregate," Highway Research Board Proceedings, 1945.

15. "Symposium on Significance of Tests of Concrete," ASTM Special Publication 169A, Second edition (1966).

16. Reagel, F. V., Gotham, D. E., "Field Observations on Effects of Joints on Cracking and Other Deterioration in Concrete Pavements," Highway Research Board Proceedings, Vol. 21, 1941.

17. Stott, J. P., Brook, K. M., "Report on a Visit to U.S.A. to Study Blowups in Concrete Roads," Road Research Laboratory Report LR 128, 1968.

APPENDIX I

DEFINITION OF PAVEMENT SURVEY TERMS AND ILLUSTRATIONS OF THESE TERMS

<u>Cracks:</u> Approximately vertical cleavage due to natural causes or traffic action. A crack across one lane is taken as one crack, a crack across two lanes as two cracks. Any crack across less than one lane is a fractional crack.

1. Transverse Cracks - Cracks which follow a course approximately at right angles to the center line.

2. Longitudinal Cracks - Cracks which follow a course approximately parallel to the centerline.

3. Diagonal Cracks - Cracks which follow a course approximately diagonal to the centerline. This cracking should not be confused with corner breaks which are local failures at the corner and generally do not extend more than 18 in. along the joint.

4. Corner Breaks - Diagonal Cracks forming an approximate isosceles triangle with a longitudinal or transverse joint, crack, or edge of slab, and has legs not more than 18 in. along the transverse joint or crack.

5. Spalling - The breaking or chipping of the pavement at joints, cracks, or edges, due to excessive shear stresses, usually resulting in fragments with feather edges.

6. Hair Checking or Cracking - Small cracks not conforming to a regular pattern, not extending to the full depth of the pavement course, and occurring before the concrete takes its final set.

7. Popout - Total dislodgement of broken or chipped areas caused by expansion of aggregate which results in craters approximately 1 to 3 in. across.

8. Surface Scaling - Peeling away of the surface of portland cement concrete, exposing sound concrete even though the scale extends into the mortar surrounding the coarse aggregate.

9. Progressive Scale - The condition of portland cement where the scaling extends below the surface stratum. Tapping or drawing a hammer over such areas generally produces a hollow or "plunky" sound.

10. Bituminous Resurfacing - Small areas of pavement resurfaced with bituminous material but at least full width or full lane.

11. Settlement - The reduction in elevation of short sections of pavement or structures due to their own weight, or the loads imposed upon them.

12. Concrete Patch - Where concrete has been replaced to its full depth.

13. Disintegration - Deterioration into small fragments or particles, usually due to some inherent fault in design, composition, construction, or maintenance.

14. Pumping - Displacement and ejection of water and suspended fine particles at pavement joints, cracks, and edges, due to accumulation of water under the pavement and movement of pavement under heavy axle loads.

15. Bituminous Patch - Smaller areas than bituminous resurfacing, or patch less than lane width.

16. Tar and Chip Patch - A small area repaired with tar and chips.

17. Pitting - The displacement of individual particles of aggregate from the pavement surface, due to the action of traffic or disintegration of the particles, without major displacement of the cementing material or mortar.

18. Flecking - The dislodgement of the thin mortar film from the outermost portions of occasional particles of coarse aggregate on a concrete surface, resulting in their exposure; generally attributable to lack of bond between the mortar and aggregate.

Figure A1. Transverse crack. In this case the crack is quite wide indicating that the longitudinal reinforcing steel has probably broken.

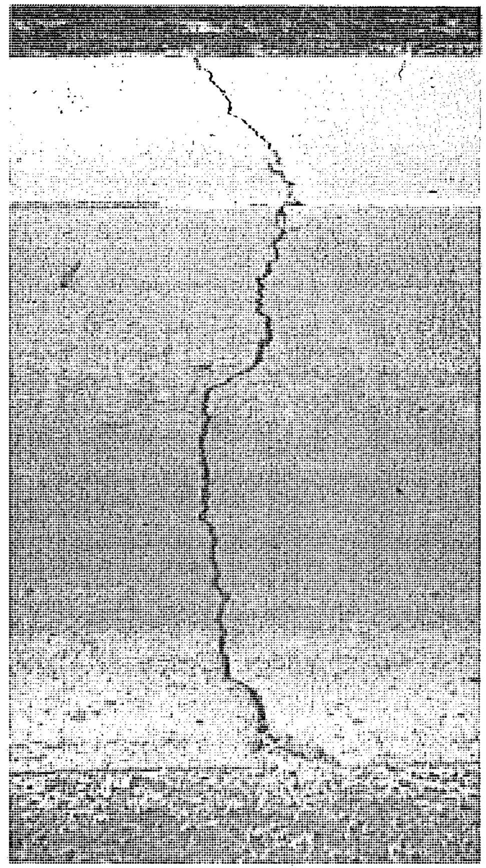


Figure A2. Typical diagonal crack.

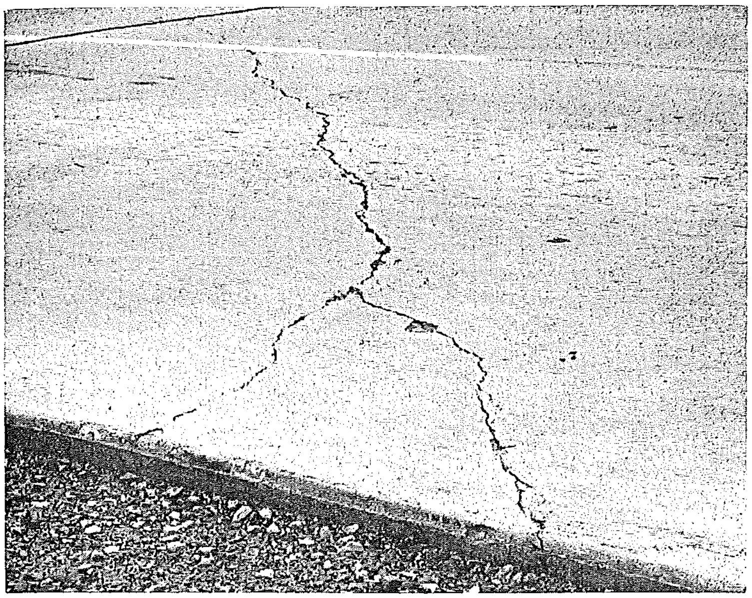


Figure A3. Longitudinal crack.

Figure A4. Infiltration crack. A short crack following a course approximately parallel to the centerline and starting from either a transverse joint or a transverse crack. Sometimes known also as a "restraint crack" or "crowfoot crack."

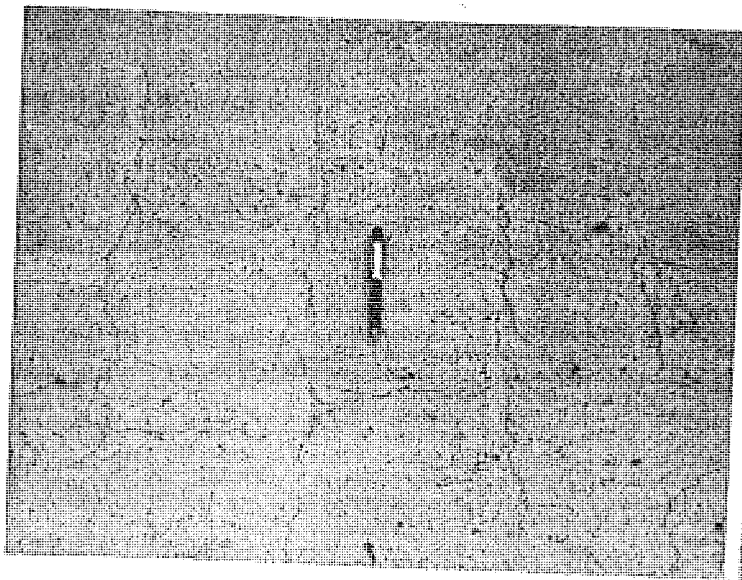
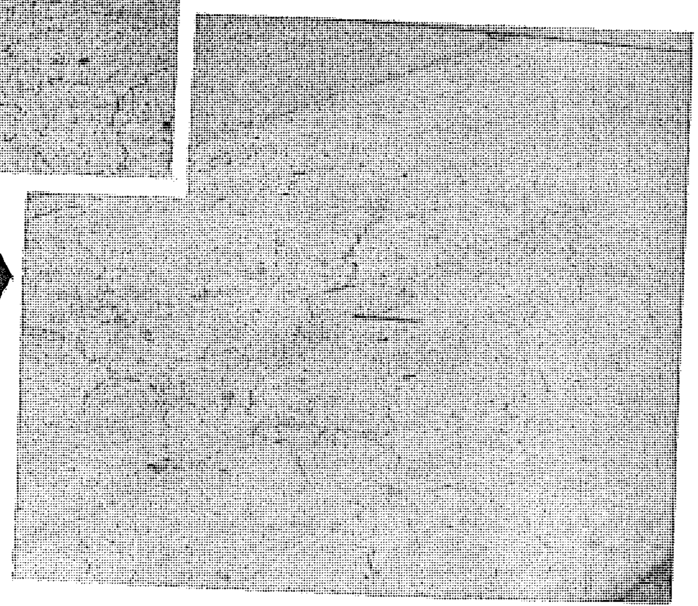


Figure A5. Map cracking. A form of disintegration in which surface cracking develops in a random pattern resembling political subdivisions on a map; may develop over the entire surface or only in localized areas; may or may not be associated with abnormal growth of the concrete.

Figure A6. Hair cracks. Small cracks occurring before the concrete takes its final set; not conforming to a regular pattern, and not extending to the full depth of the pavement slab. Sometimes termed "hair cracking" or "hairline cracking."



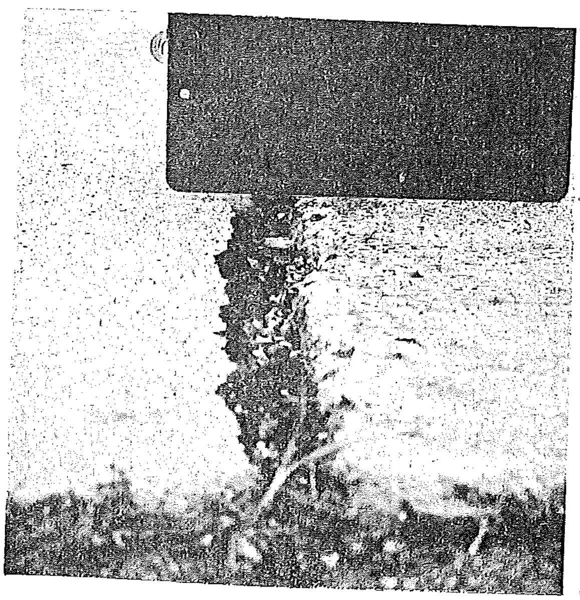


Figure A7. Faulting. The differential vertical displacement of the slabs adjacent to a joint or crack.

Figure A8. Joint blowup. The localized buckling or shattering of a rigid pavement at a joint caused by excessive longitudinal pressure.

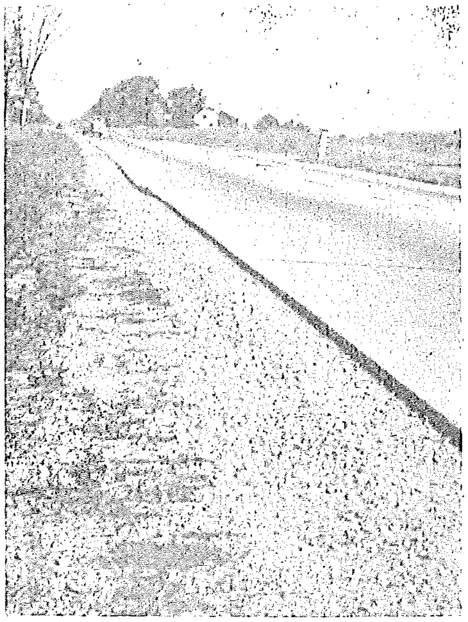


Figure A9. Settlement. Reduction in elevation of short sections of pavement or structures due to their own weight, to the loads imposed on them, or to shrinkage of the supporting soil.

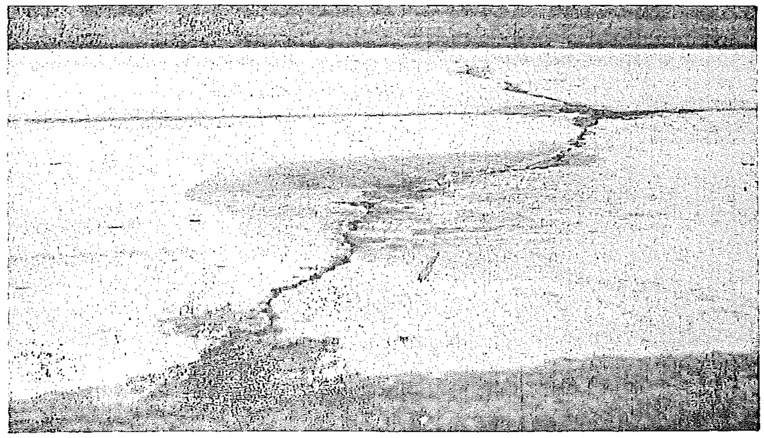
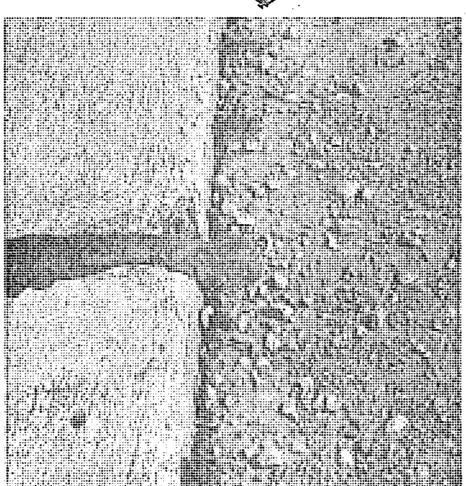


Figure A10. Pumping. The ejection of water or mixtures of water and clay or silt along transverse joints and cracks and along pavement edges caused by downward slab movement activated by the passage of heavy axles over the pavement, after the accumulation of free water in the subgrade or subbase. Sometimes termed "mud pumping" or "water pumping."

Figure A11. Blowing. The ejection of sand or dust along transverse or longitudinal joints or cracks or along pavement edges; the results of air pressure caused by downward slab movement activated by the passage of heavy axles over the pavement.



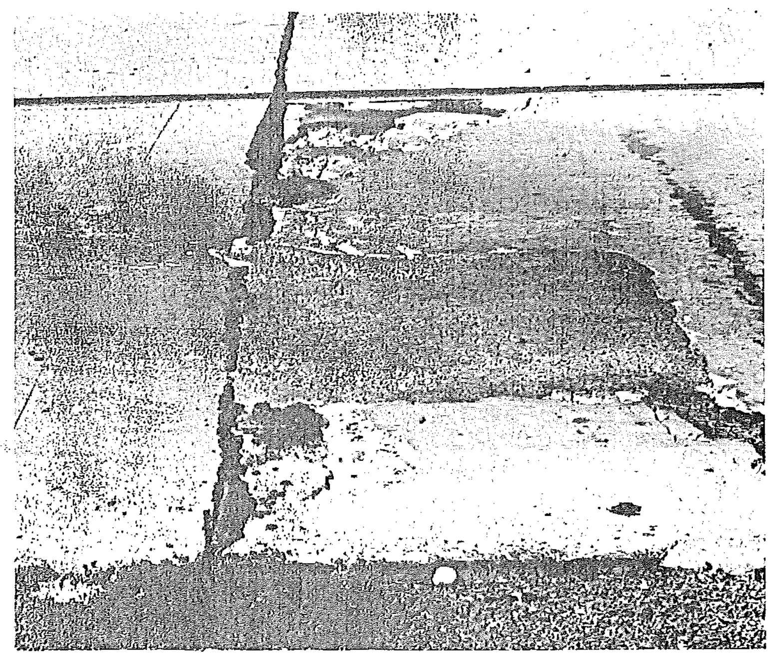


Figure A12. Patch. In an area less than a full lane in width, (a) the covering over with or without removal of the existing pavement, or (b) removal and replacement of all the existing pavement, with portland cement concrete, bituminous concrete, tar and chip mix, or other materials.

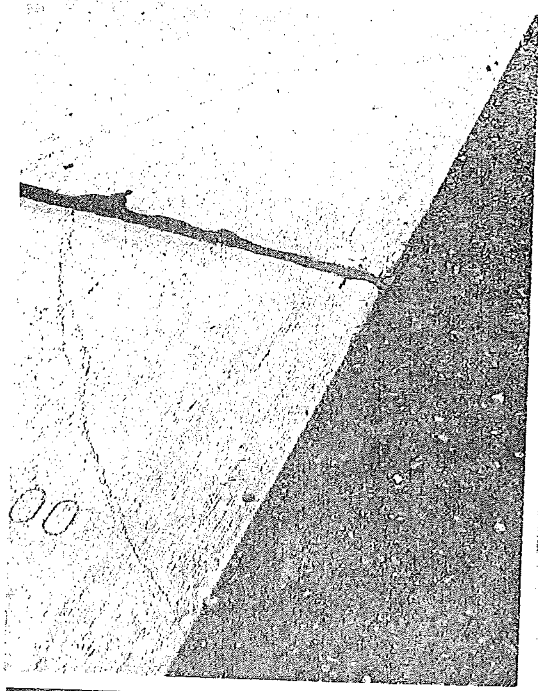


Figure A13. Corner break. A vertical fracture partially or completely through the slab, forming an approximate isosceles triangle with a transverse joint or crack and the outer or longitudinal joint; involves not more than half a lane width. Various types of corner breaks are as follows: corner break at joint, external or internal; corner break at crack, external or internal. The one shown is at a joint and is external.

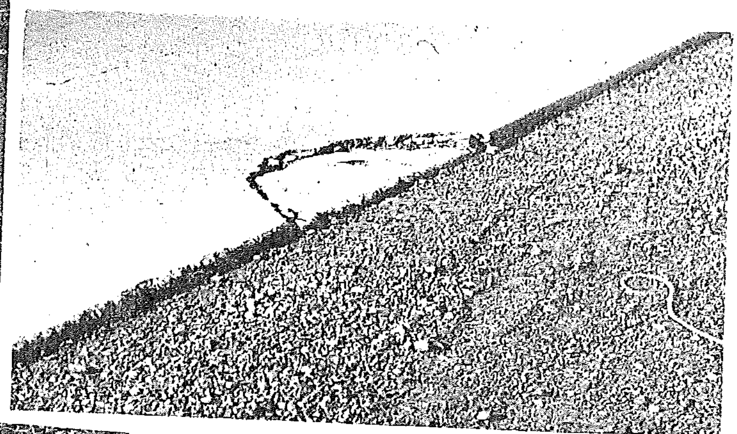
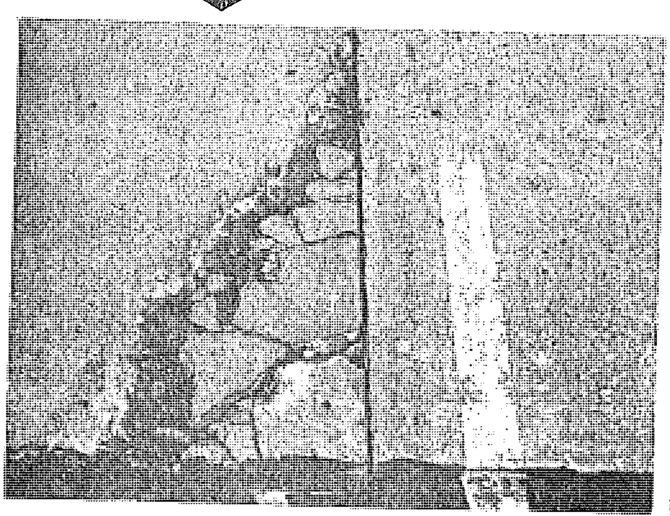


Figure A14. Edge break.

Figure A15. Slab corner spalling, exterior.

Figure A16. Slab corner spalling, interior.



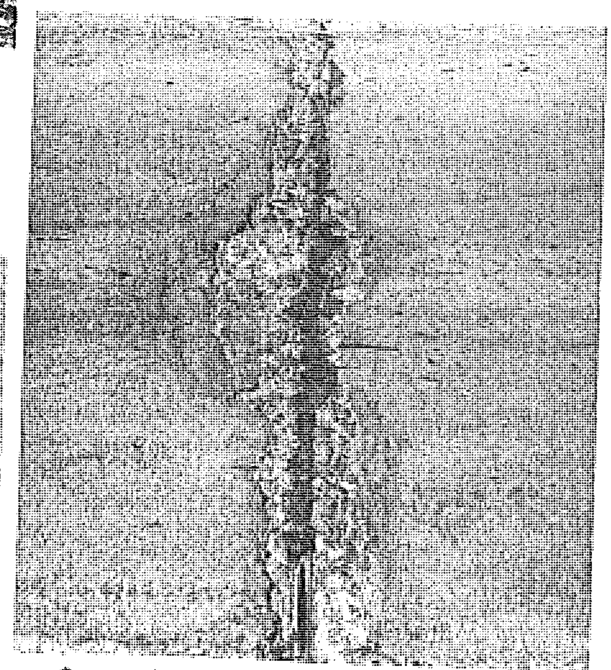


Figure A17. Joint spalling, interior.

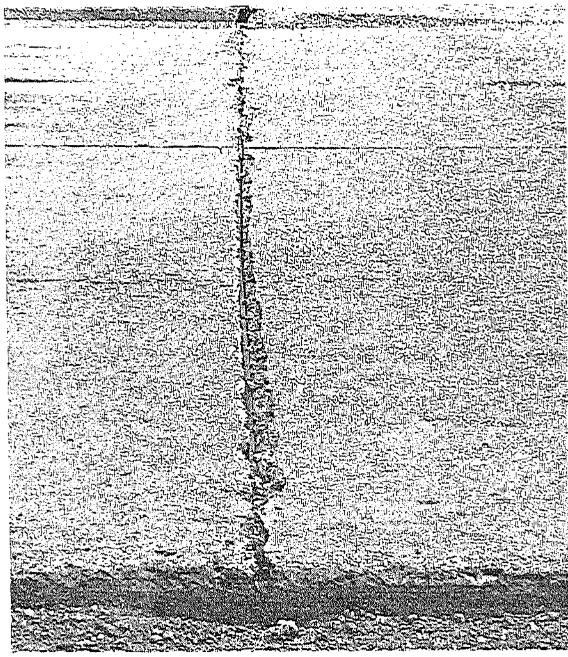
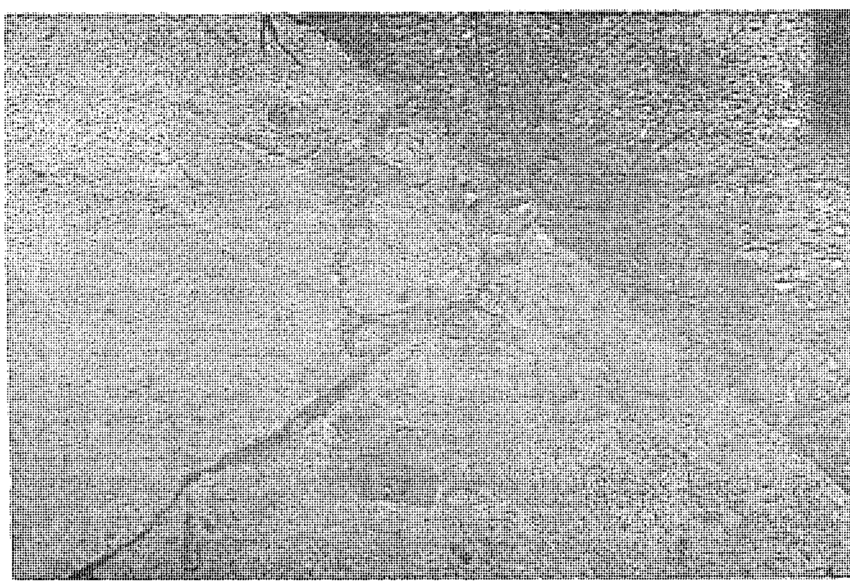


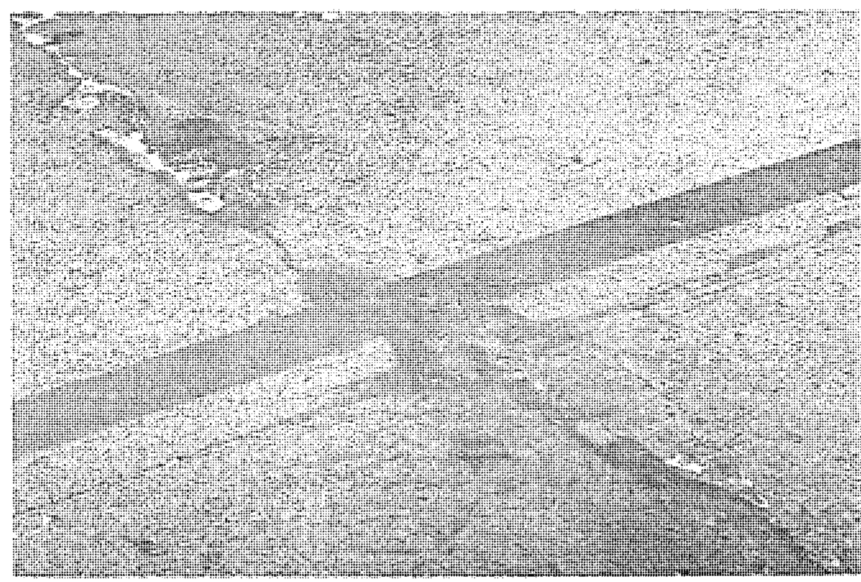
Figure A18. Joint groove spalling.

Figure A19. Outer edge spalling.

Figure A20 (bottom left). Crack spalling, exterior.

Figure A21 (bottom right). Crack spalling, interior.





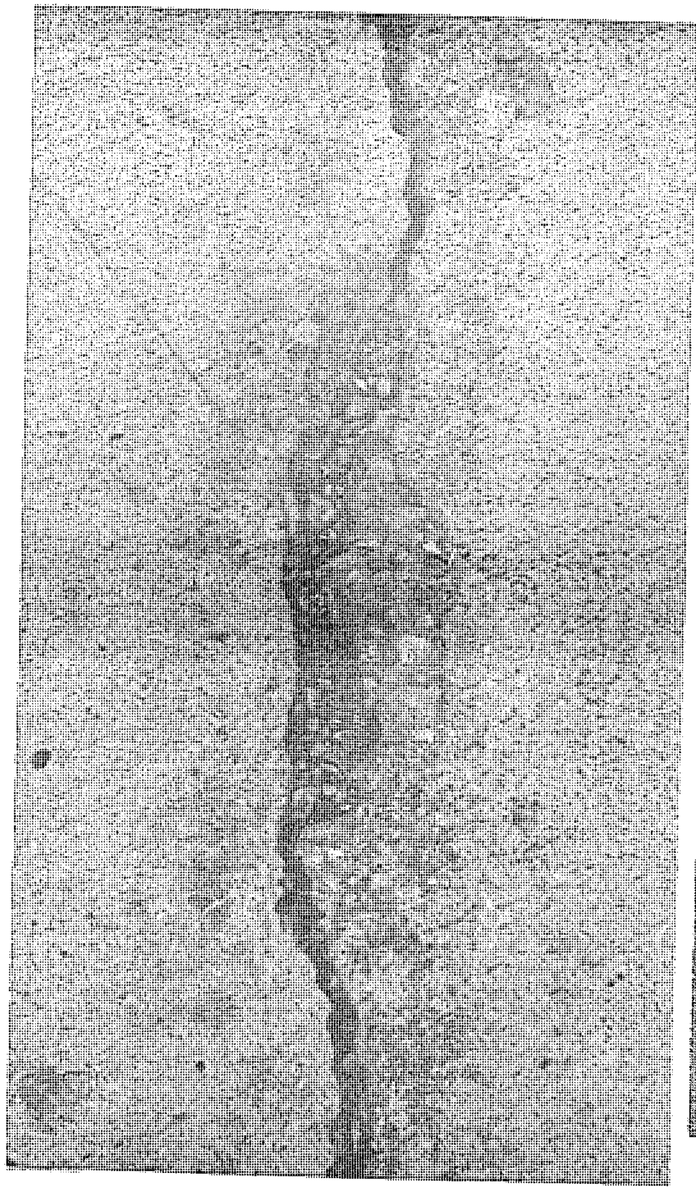


Figure A22. Spalling along crack.

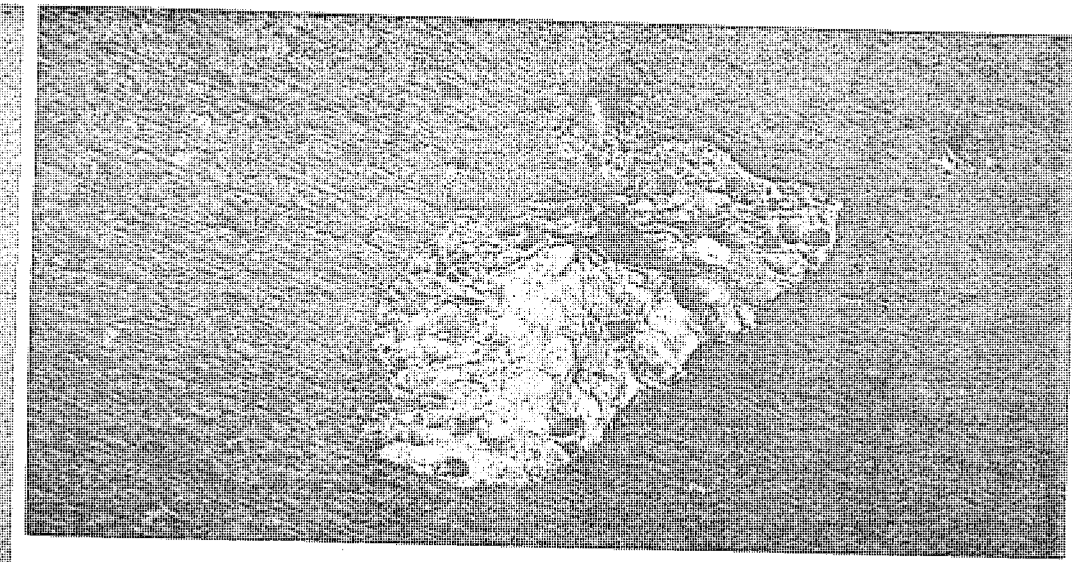


Figure A23. Surface spalling.

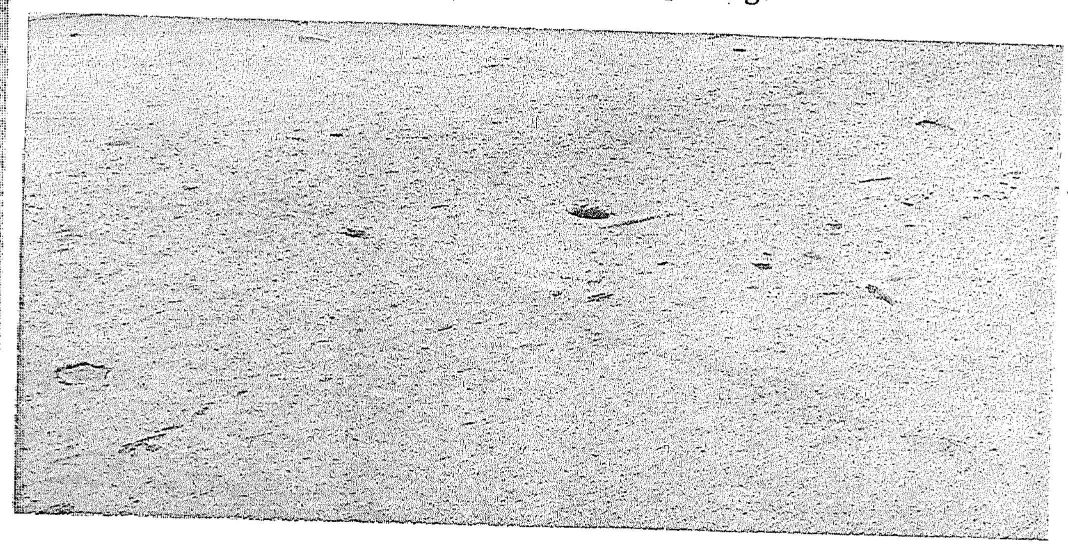


Figure A24. Mudjacking. A treatment to correct settlement of pavements. Holes are bored in the pavement and suitable materials pumped under the slab to raise it to the desired elevation.

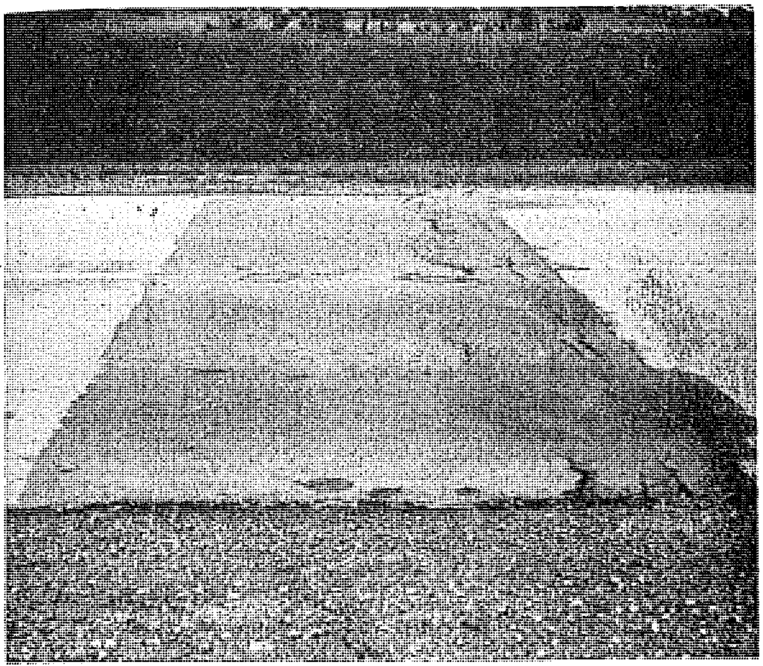


Figure A25. Resurfacing. In an area at least a full lane in width, (a) the covering-over with or without the removal of the existing pavement, or (b) removal and replacement of all existing pavement with portland cement concrete, bituminous concrete, tar and chip mix, or other materials.

Figure A26. Disintegration. Deterioration into small fragments or particles, usually due to some inherent fault in design, composition, construction, or maintenance.

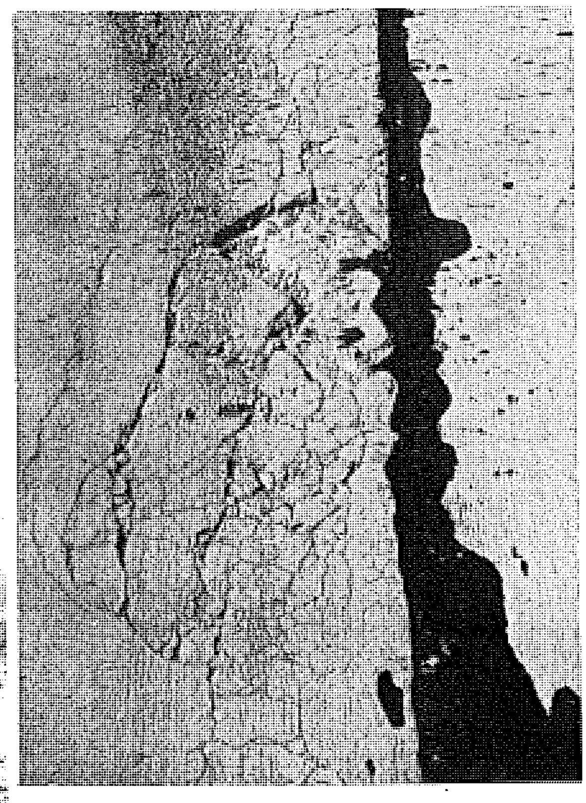


Figure A27. Frost heave. The differential upward displacement of pavement due to action of frost which has caused localized swelling of the subgrade.

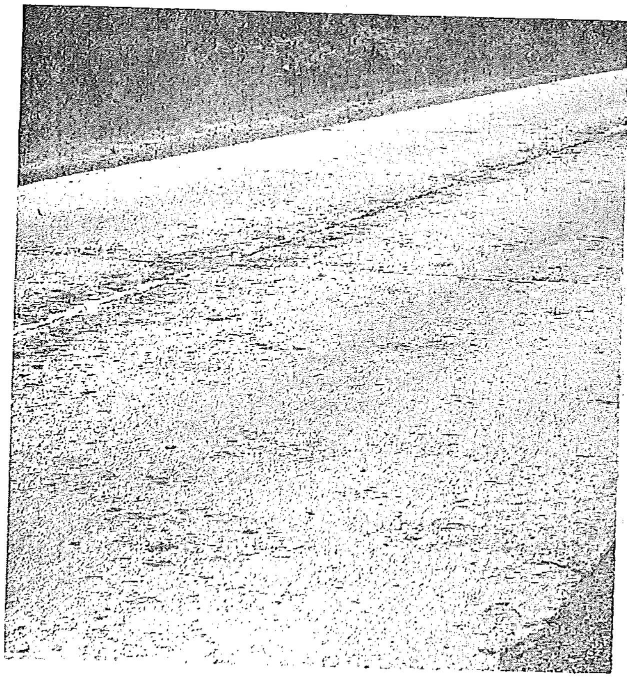


Figure A28. Surface scale. The peeling away of surface mortar exposing sound concrete, even though the scale extends into the mortar surrounding the coarse aggregate.

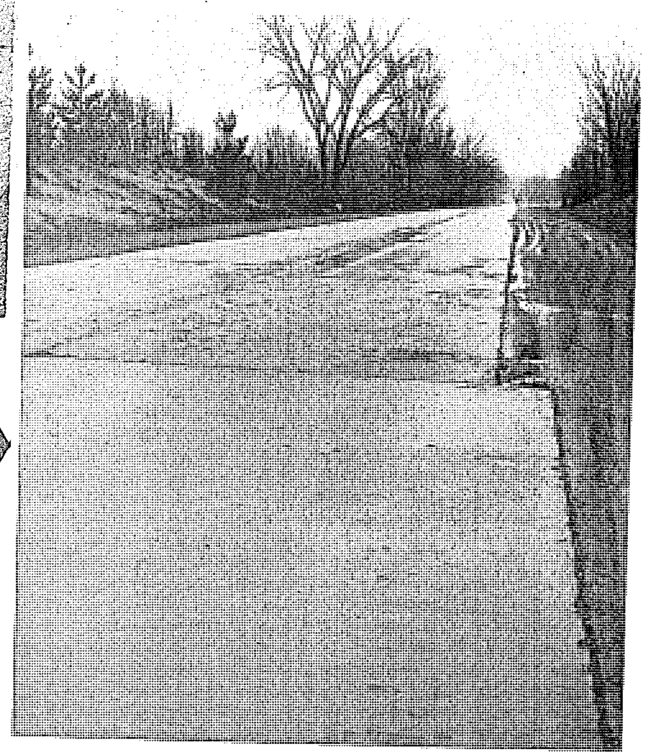


Figure A29. Progressive scale. A condition of concrete disintegration which in its initial stages appears as surface scale, but gradually progresses deeper below the surface stratum. Tapping or drawing a hammer over such areas generally produces a hollow or "plunky" sound.

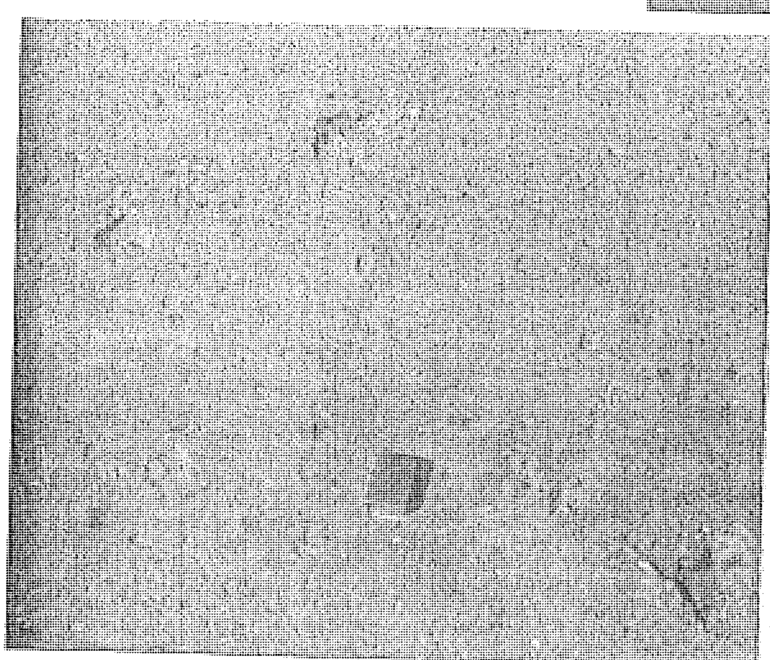


Figure A30. Pitting. The displacement of individual aggregate particles from the pavement surface, due to the action of traffic or disintegration of the particles, without major displacement of the cementing material or mortar.

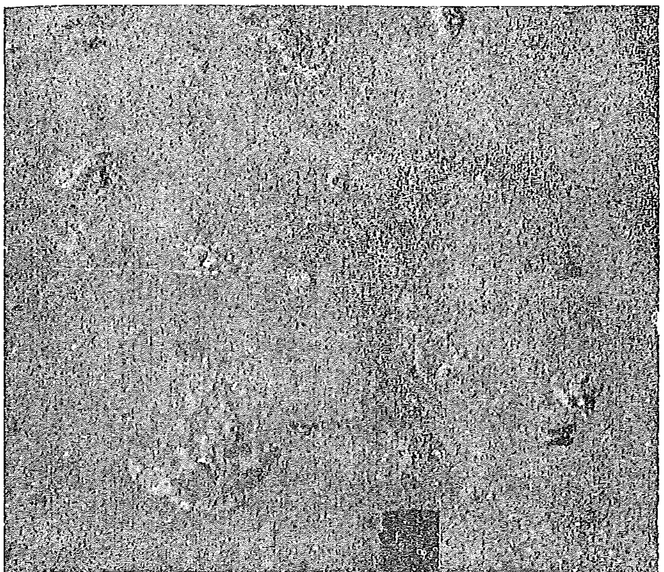
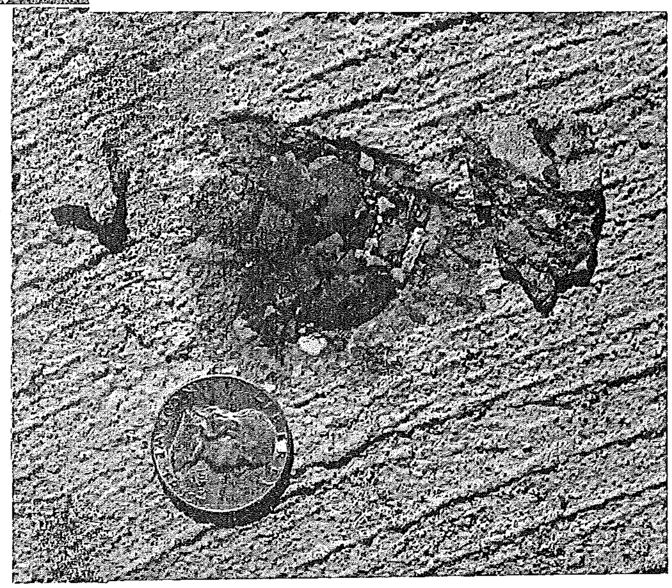


Figure A 31. Flecking. Dislodging of the thin mortar film over coarse aggregate particles near the pavement surface, resulting in exposure of the particles; generally attributed to a lack of bond between the mortar and aggregate. This is generally an early stage of pitting and although limited in extent resembles surface scaling.

Figure A32. Pop-out. A crater-like depression generally 1 to 3 inches in diameter caused by the breaking away or forcing up of a portion of the slab surface, due to expansion of a piece of underlying coarse aggregate; associated with soft, light-weight, porous aggregate such as chert.



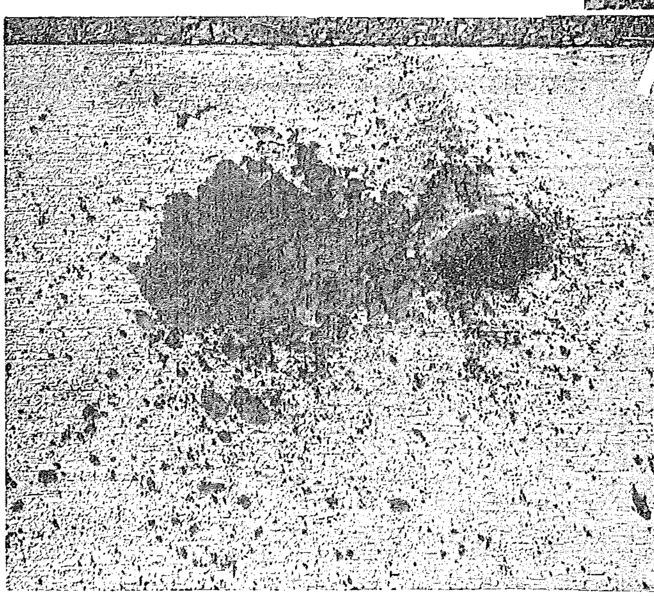


Figure A33. Clay pocket. A hole in the pavement resembling a pop-out, caused by a lump of clay in the aggregate used in construction; the result of disintegration under freeze-thaw conditions and traffic.

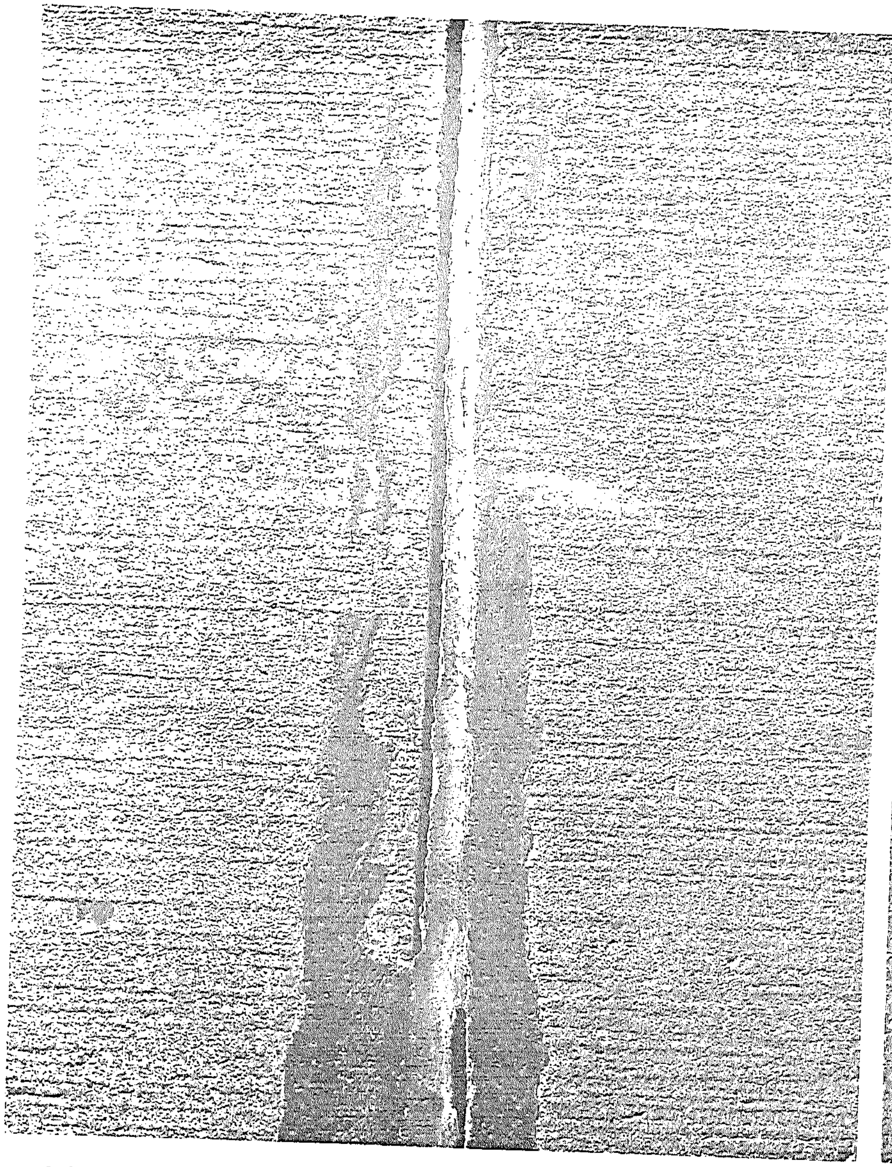


Figure A34. Joint seal adhesion failure. A bond failure between joint seal and the face of the joint seal groove.

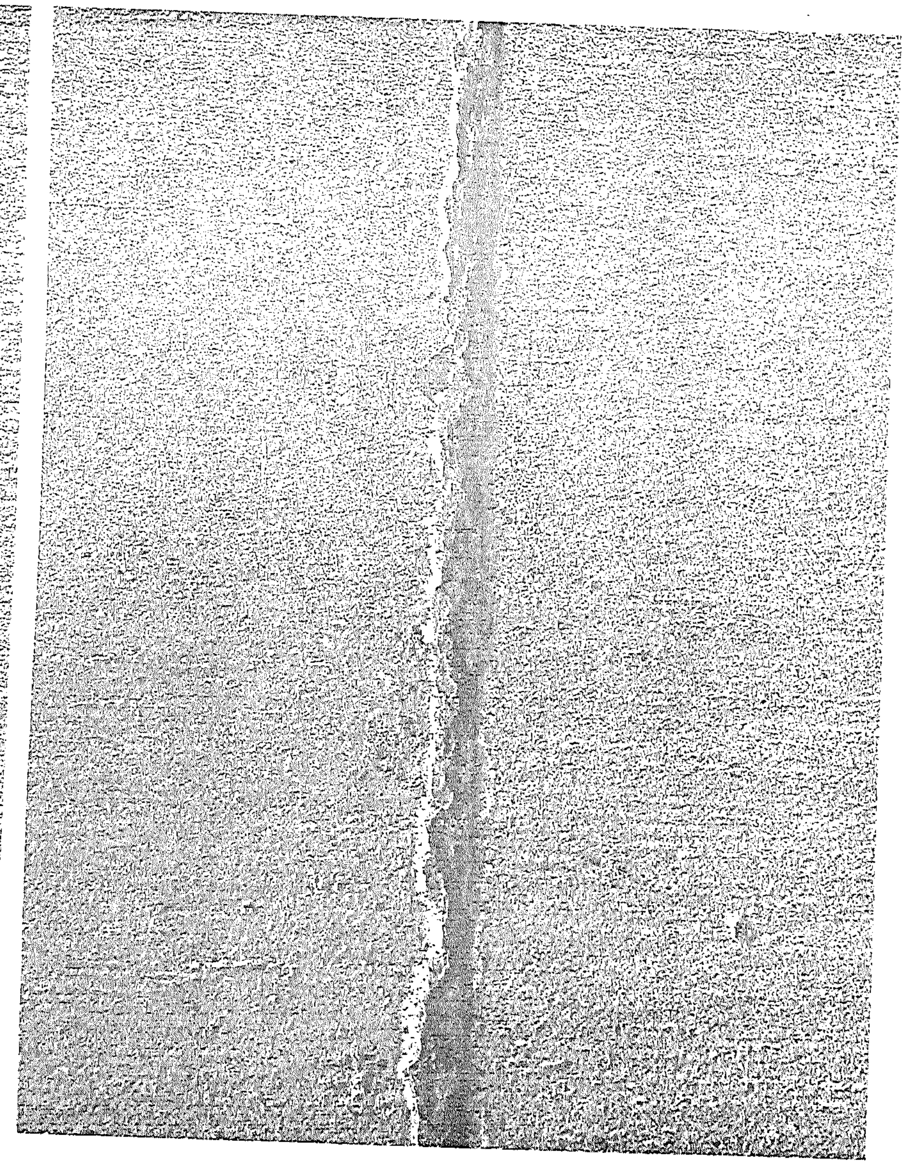


Figure A35. Joint seal cohesion failure. A bond failure within the joint seal itself.

APPENDIX II

FULL STATISTICAL ANALYSIS OF VARIANCE FOR PRF AND PSI DATA

TABLE B-1
ANALYSIS OF VARIANCE FOR 5-, 10-, & 15-YR
(Divided Expressway)

Comparison	Source of Variation	DF	Expected Mean Square	Five Years		Ten Years		Fifteen Years	
				Mean Square	F	Mean Square	F	Mean Square	F
Between Replicate, Error	R (Roadway)	1	$\sigma^2 + \sigma^2_R$	0.08		0.06		1.18	
Within Replicate, Between Contract	S (% soft)	1	$\sigma^2 + \sigma^2_{BC} + 8\theta^2_S$	0.01	0.0	4.27	6.4	15.80	12.4*
	SR	1	$\sigma^2 + \sigma^2_{BC}$	0.59		1.24		0.83	
	T (Traffic)	1	$\sigma^2 + \sigma^2_{BC} + 8\theta^2_T$	0.34	0.8	17.16	25.6*	5.36	4.2
	T x R	1	$\sigma^2 + \sigma^2_{BC}$	0.65		0.02		2.86	
	S x T	1	$\sigma^2 + \sigma^2_{BC} + 4\theta^2_{ST}$	0.69	1.5	0.23	0.3	4.58	3.6
	S x T x R	1	$\sigma^2 + \sigma^2_{BC}$	0.11		0.74		0.11	
	Avg. Error	3	$\sigma^2 + \sigma^2_{BC}$						
Between Lane, Within Contract	L (lane)	1	$\sigma^2 + \sigma^2_{BL} + 8\theta^2_L$	0.06	0.8	5.14	17.7*	8.07	161.4*
	L x R	1	$\sigma^2 + \sigma^2_{BL}$	0.00		0.45		0.00	
	L x S	1	$\sigma^2 + \sigma^2_{BL} + 4\theta^2_{LS}$	0.13	1.6	0.00	0.0	0.93	18.6*
	L x S x R	1	$\sigma^2 + \sigma^2_{BL}$	0.01		0.26		0.07	
	L x T	1	$\sigma^2 + \sigma^2_{BL} + 4\theta^2_{LT}$	0.08	1.0	2.15	7.4	2.00	40.0*
	L x T x R	1	$\sigma^2 + \sigma^2_{BL}$	0.23		0.35		0.03	
	L x S x T	1	$\sigma^2 + \sigma^2_{BL} + 2\theta^2_{LST}$	0.03	0.4	0.45	1.6	0.31	6.2
	L x S x T x R	1	$\sigma^2 + \sigma^2_{BL}$	0.07		0.08		0.11	
	Avg. Error	4	$\sigma^2 + \sigma^2_{BL}$						

* Asterisk is used if effects are statistically significant at the 0.05 level.

TABLE B-2
ANALYSIS OF VARIANCE FOR 10-, & 15-YR
(Divided Expressway)

Comparison	Source of Variation	DF	Expected Mean Square	Ten Years		Fifteen Years	
				Mean Square	F	Mean Square	F
Between Replicate, Error	R (Roadway)	1	$\sigma^2 + \sigma^2_R$	.0002		.0218	
Within Replicate, Between Contract	S (% soft)	1	$\sigma^2 + \sigma^2_{BC} + 8\theta^2_S$	.1425	8.0	.2377	31.6*
	S x R	1	$\sigma^2 + \sigma^2_{BC}$	.0005		.0189	
	T (Traffic)	1	$\sigma^2 + \sigma^2_{BC} + 8\theta^2_S$	.0116	0.7	.0452	6.0
	T x R	1	$\sigma^2 + \sigma^2_{BC}$	.0281		.0014	
	S x T	1	$\sigma^2 + \sigma^2_{BC} + 4\theta^2_{ST}$	.1785	10.0	.0743	9.9
	S x T x R	1	$\sigma^2 + \sigma^2_{BC}$	.0248		.0023	
	Avg. Error	3	$\sigma^2 + \sigma^2_{BC}$	.0178		.0075	
Between Lane, Within Contract	L (lane)	1	$\sigma^2 + \sigma^2_{BL} + 8\theta^2_L$	.0008	0.7	.0060	0.8
	L x R	1	$\sigma^2 + \sigma^2_{BL}$	.0005		.0218	
	L x S	1	$\sigma^2 + \sigma^2_{BL} + 4\theta^2_{BL}$	.0003	0.3	.0046	0.6
	L x S x R	1	$\sigma^2 + \sigma^2_{BL}$	.0028		.0005	
	L x T	1	$\sigma^2 + \sigma^2_{BL} + 4\theta^2_{LT}$	.0086	7.7	.0023	0.3
	L x T x R	1	$\sigma^2 + \sigma^2_{BL}$	.0003		.0060	
	L x S x T	1	$\sigma^2 + \sigma^2_{BL} + 2\theta^2_{LST}$	.0005	0.5	.0005	0.1
	L x S x T x R	1	$\sigma^2 + \sigma^2_{BL}$	.0008		.0014	
	Avg. Error	4	$\sigma^2 + \sigma^2_{BL}$				

* Asterisk is used if effects are statistically significant at the 0.05 level.