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.

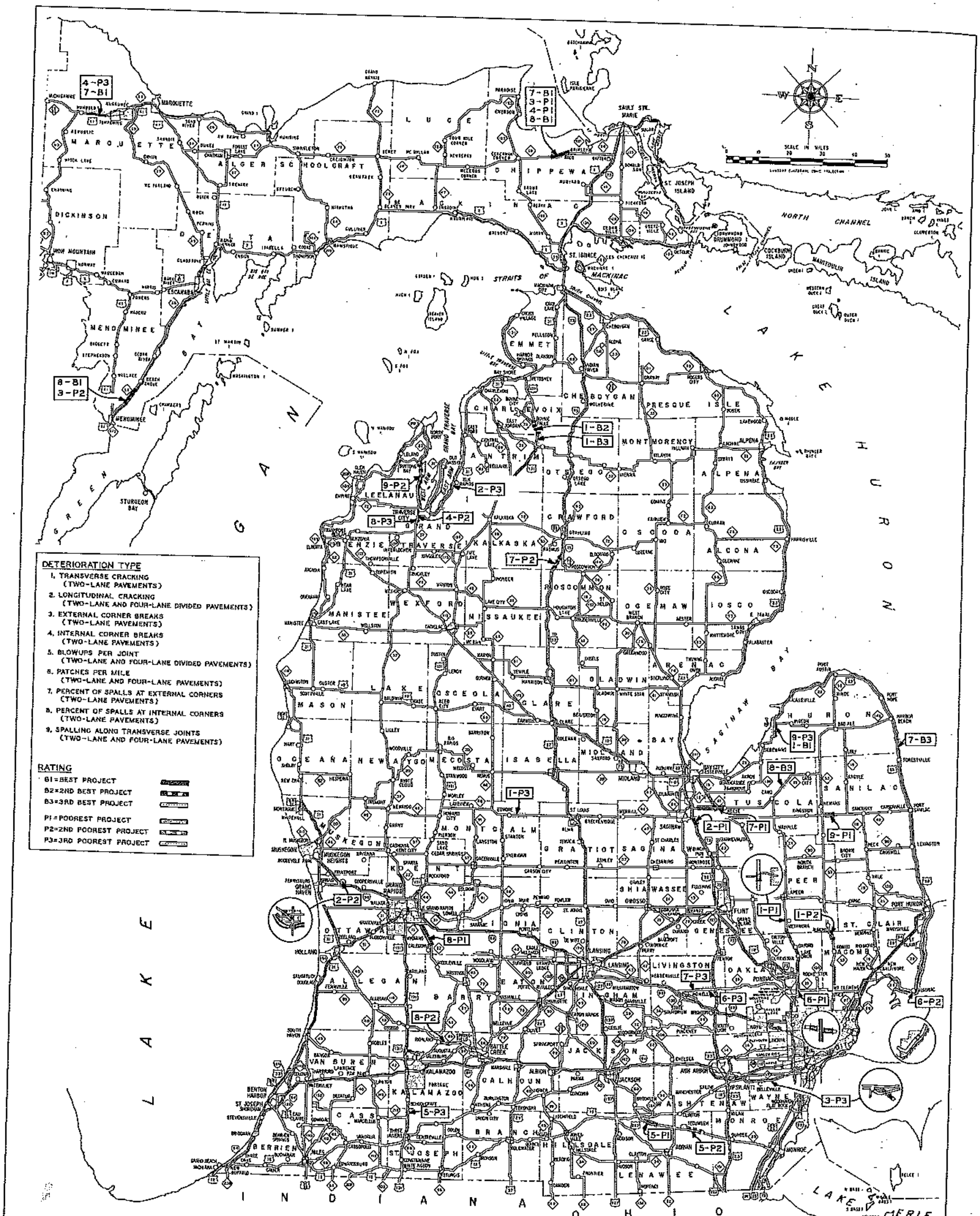


Figure 71. Location of best and poorest performing projects for 9 types of pavement deterioration (10 years of service).

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:

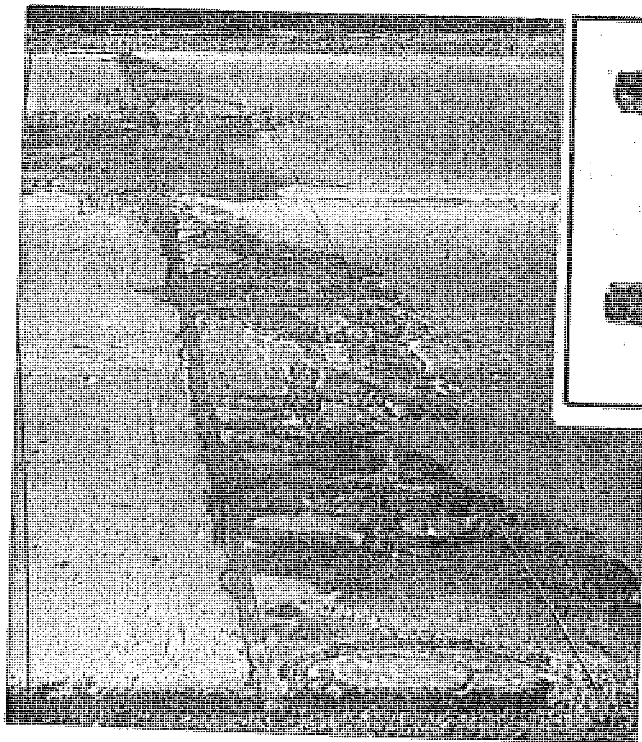
1. The metal expansion caps, installed on the end of dowels to permit pavement expansion, were never installed. Therefore, the compressive force on the end of the dowel shattered the concrete and bent the bar (Fig. 72).

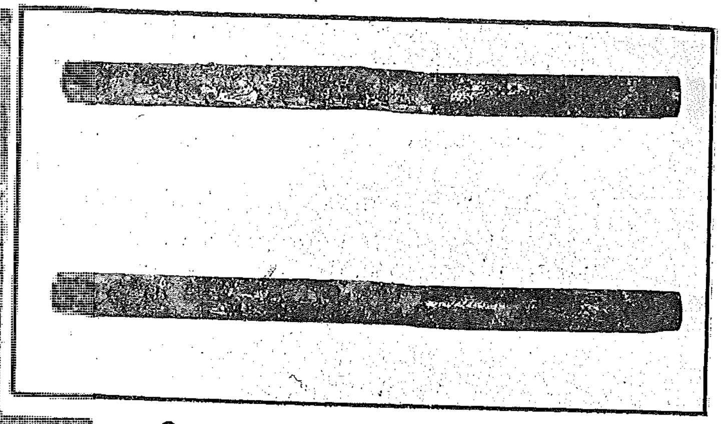
2. Some of the dowels were badly misaligned both horizontally and vertically.

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.

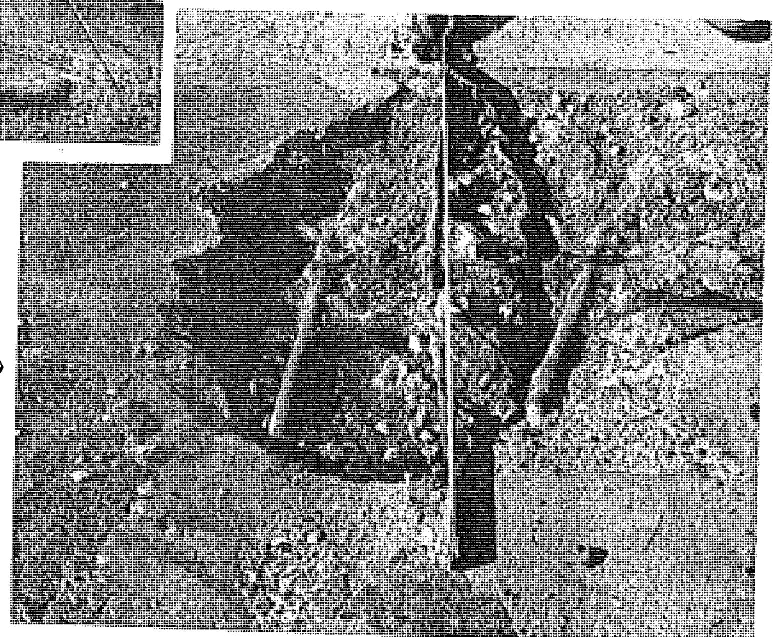
There were also contraction joints on this same project that showed distress but it was not possible to examine them since maintenance forces had repaired them before we were notified of the problem. Of the four observations noted for the expansion joint, the second and third (dowel misalignment and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



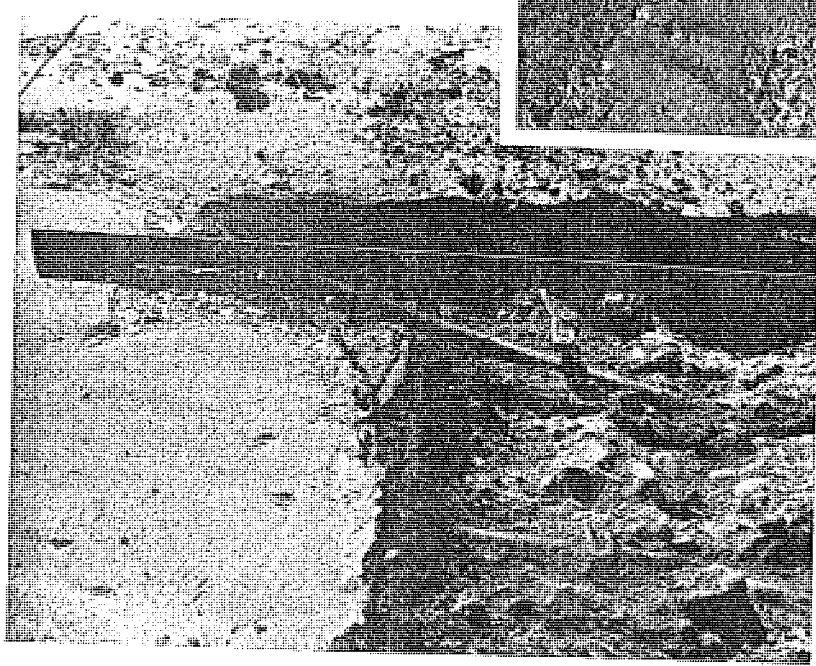


Physical condition of two dowels; note bending due to compressive force.

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.

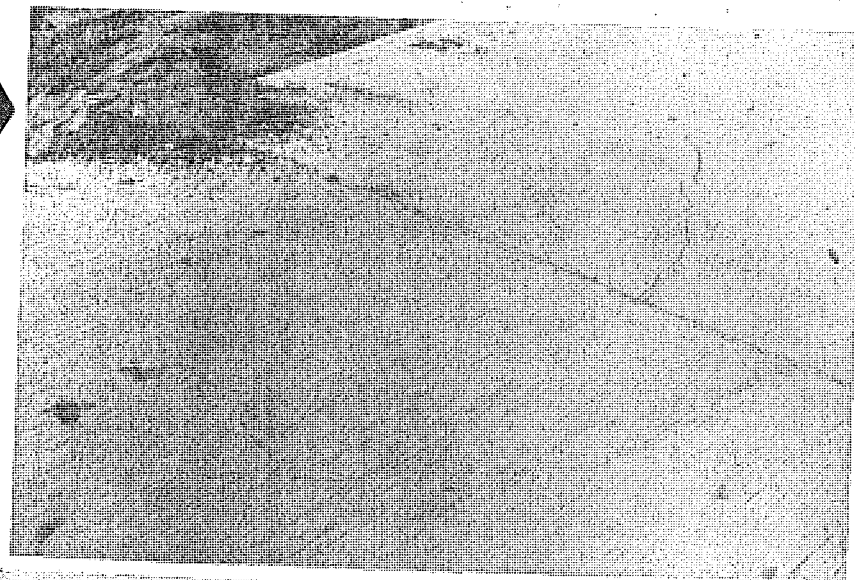
In 1959 the Research Laboratory summarized joint problems that had developed over the past few years. These included: 1) longitudinal cracking at the joint, 2) joint spalling, 3) joint blowups, 4) inadequate load-transfer assemblies, 5) dowel bar corrosion, 6) concrete failure at construction joints,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

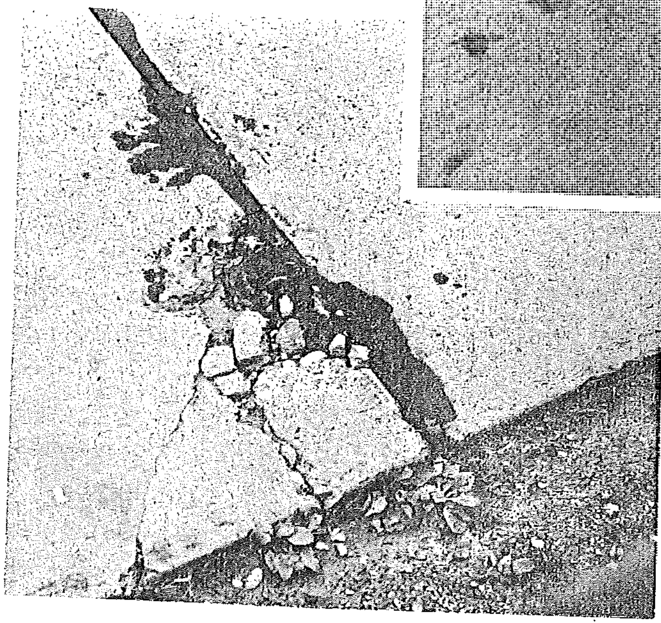
1. Heavy loads during early life of the structure, such as earth-moving machinery or other heavy contractor's equipment. (This is especially true for sawed longitudinal joints in comparison with the previous practice of a premolded bituminous strip to form the longitudinal plane-of-weakness joint, since 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.

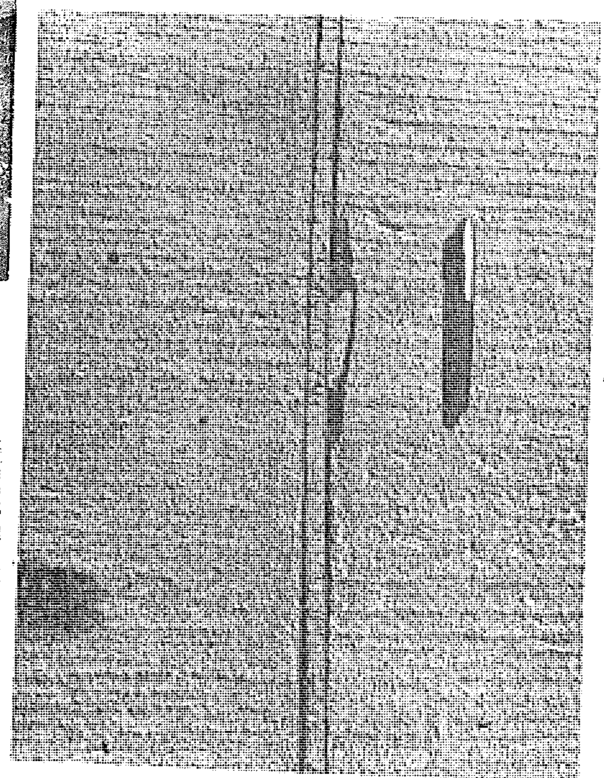
Longitudinal cracking at the joint.



Concrete spalling; external corner.



Joint groove spalling.



Concrete spalling; interior corner.

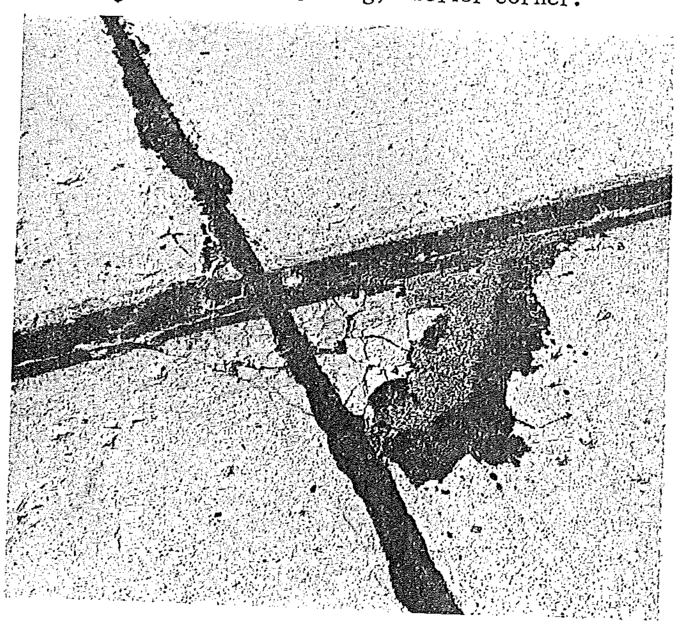
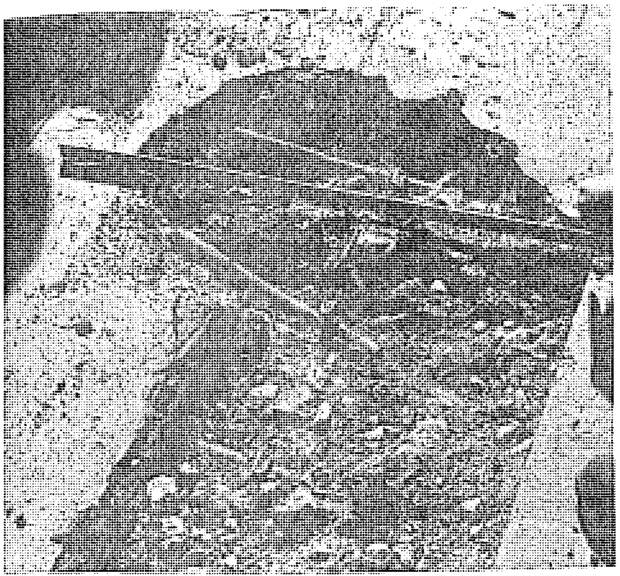
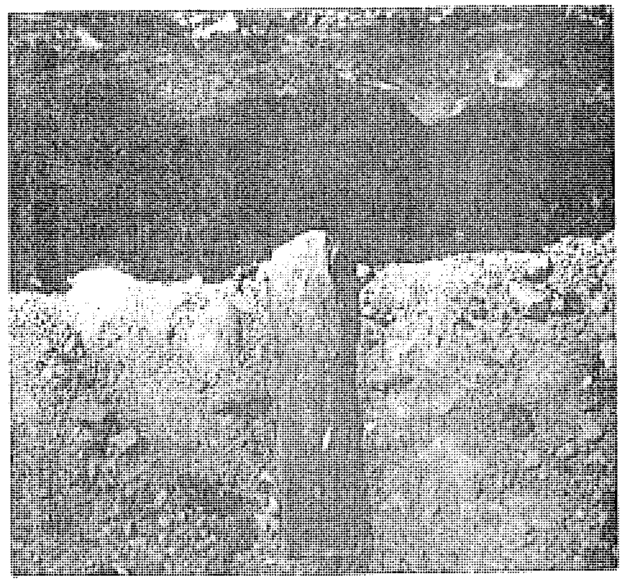


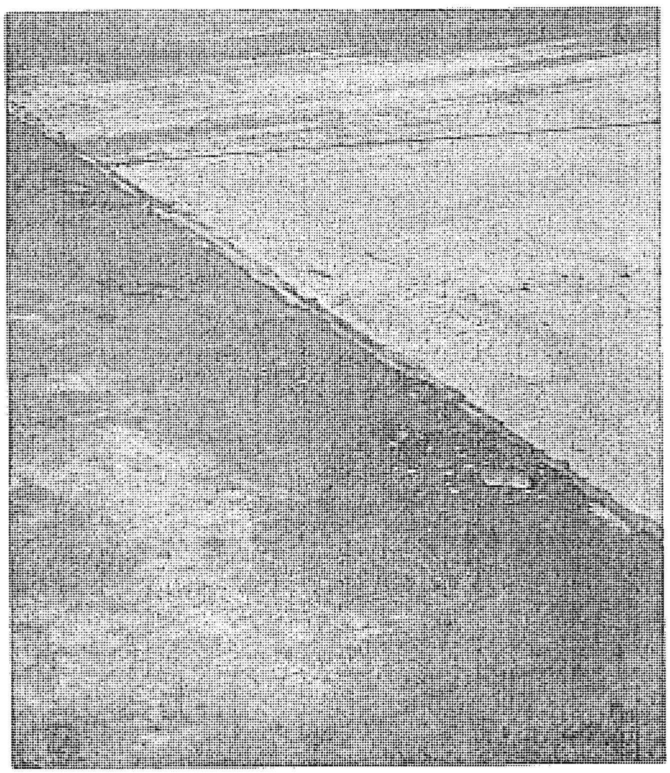
Figure 73. Two joint problems. Longitudinal cracking and joint spalling.



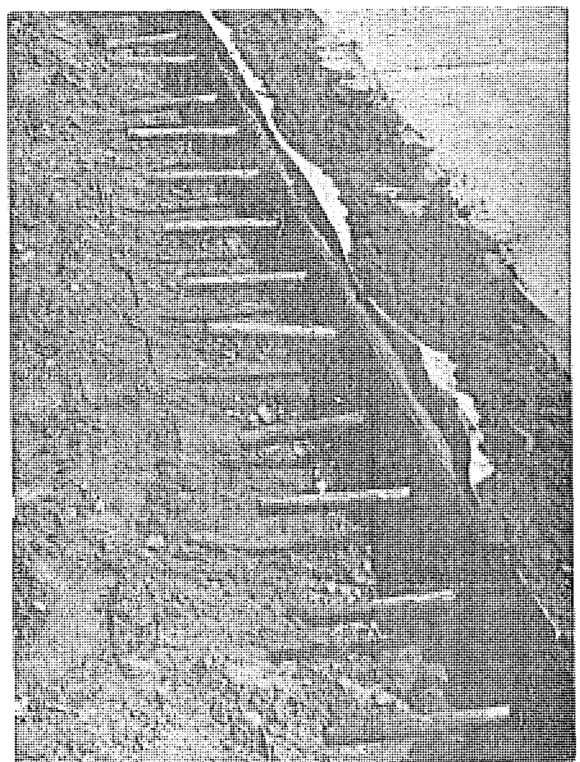
Dowel misalignment during concrete placement due to inability of assembly to hold dowels in place.



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.

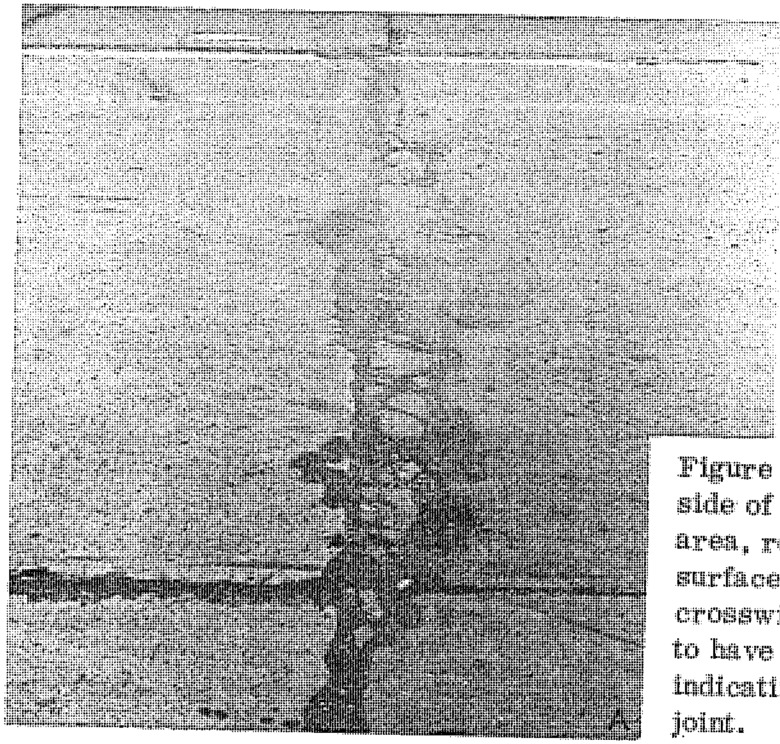
However, one further experimental project in joint sealing was attempted to determine if the shape factor of the joint groove, as presented by Professor Egon Tons in 1959, and somewhat shorter slab lengths would result in satisfactory joint seal performance. On the project, five groove sizes were tried, 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.

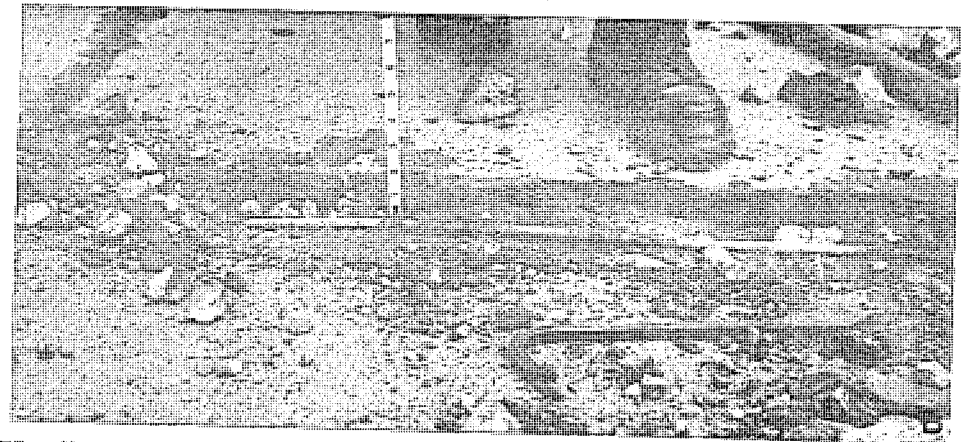
As a result of the Department's experience over more than ten years, namely that liquid-type joint seals were incapable of properly sealing transverse joints for longer slabs, it was very receptive to the use of preformed neoprene when this material was developed several years ago. It was first experimentally 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.

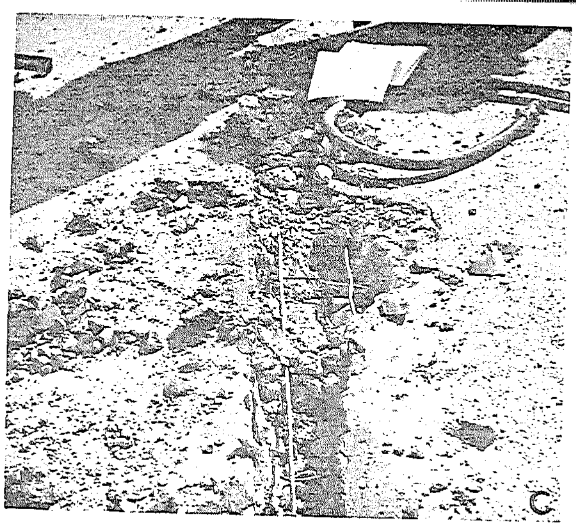
In 1960 joint spalling on an urban expressway in the western side of the State was called to our attention by maintenance personnel. The three most severely spalled joints were exposed and examined prior to repair to determine the cause of the deterioration. The first joint evidenced extensive surface 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.

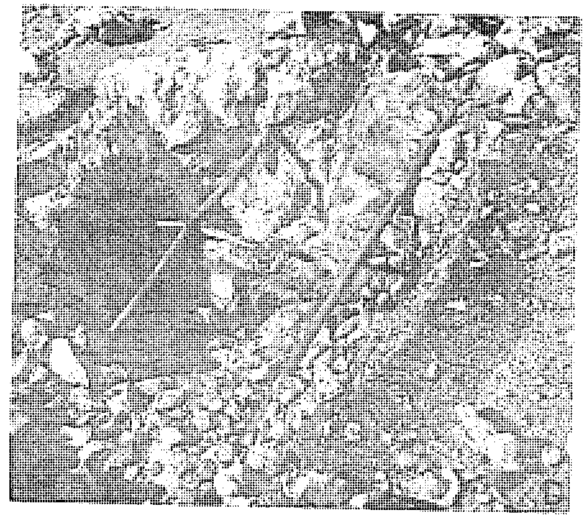
The second joint was a construction joint. Where the concrete was spalled most severely down to the levels of the dowels it was noted that all four dowels exposed tilted up 1/4 to 1/2 in. This misalignment is sufficient to the cause concrete-to-dowel binding and appears to have caused the spalling at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









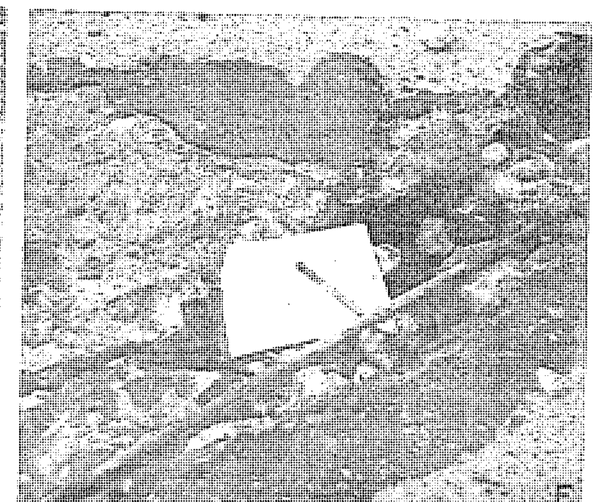


Figure 75. "Joint 3" where opening had occurred about 6 in. on the spalled side of the joint groove, is shown in an overall view (A). In the spalled area, reinforcement was embedded only 7/8- to 1 in. beneath the concrete surface (B). The reinforcing mat here was found to have been placed crosswise at 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

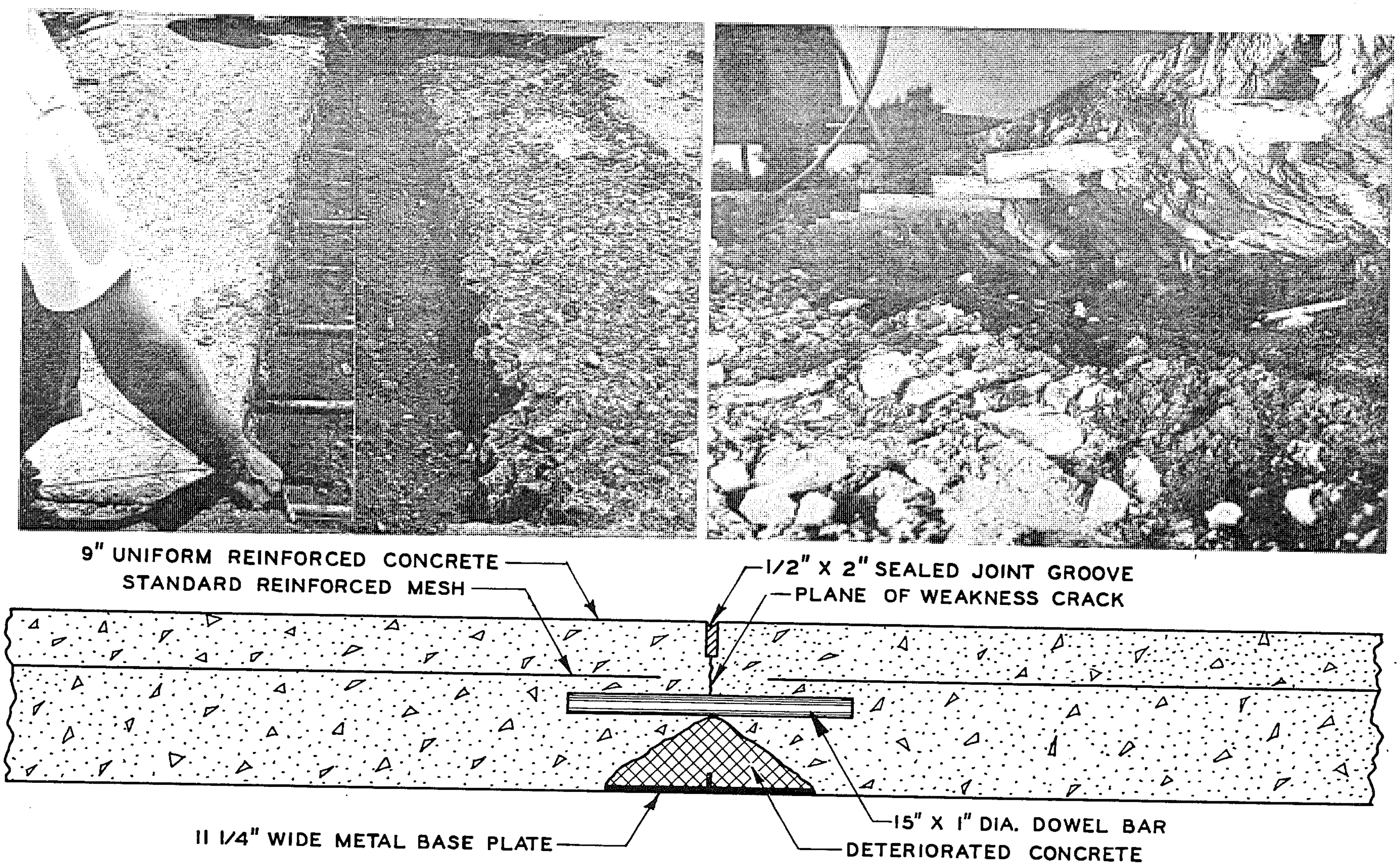


Figure 76. Pavement joint blow-up, dowel and joint groove misalignment observed during repair (upper left), concrete deterioration found below dowel bars (upper right) and schematic cross-section of deterioration.

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

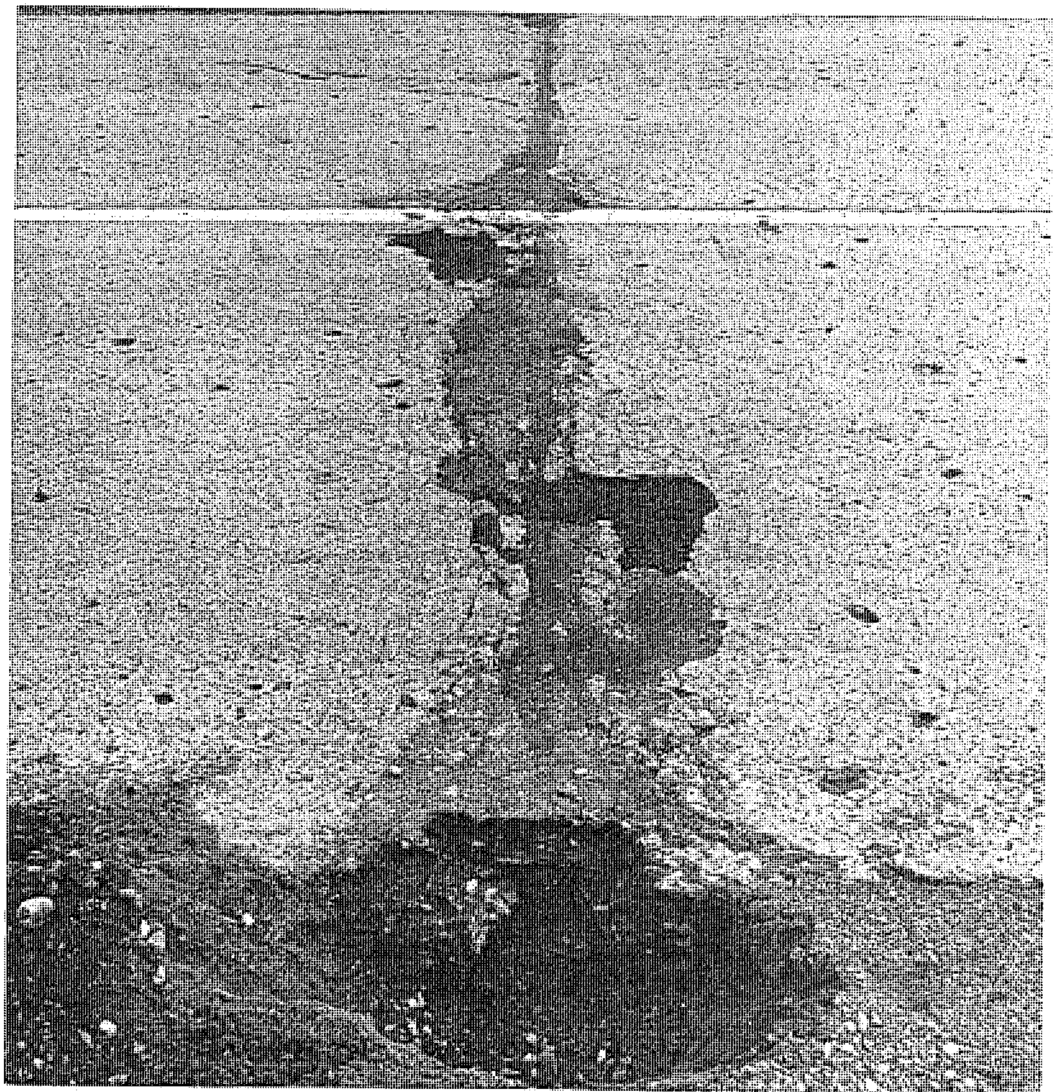


Figure 77. Distressed joint with end exposed.

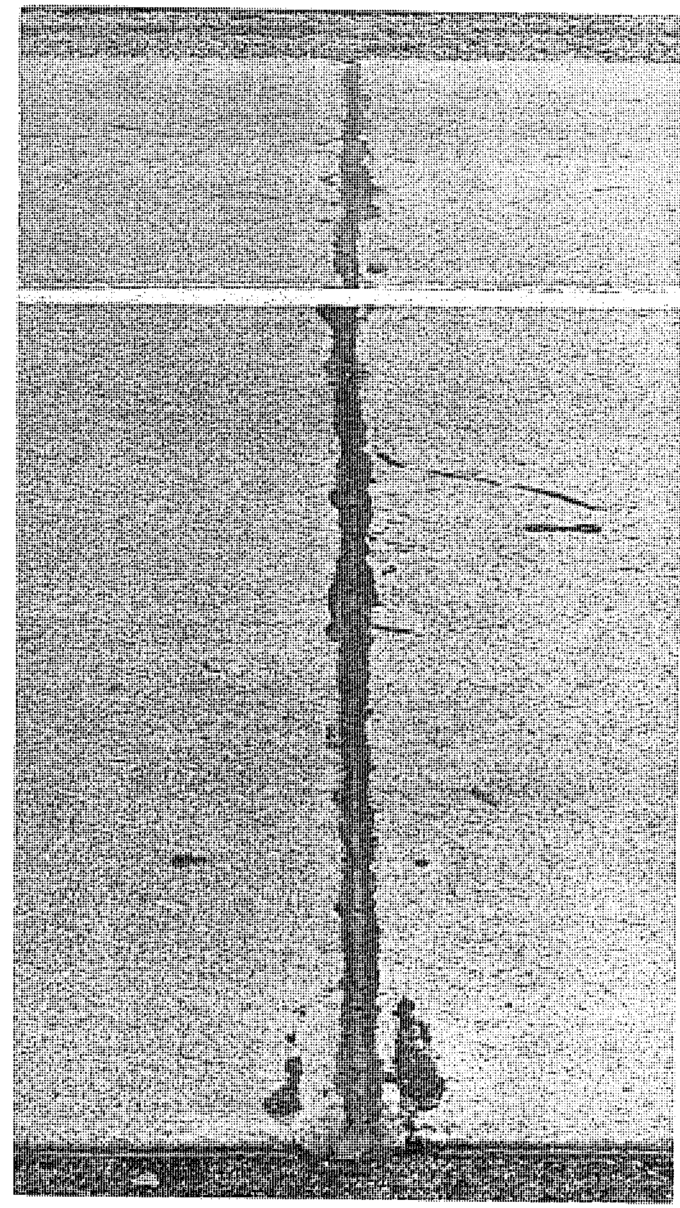


Figure 78. Typical joint in good performance area.

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.

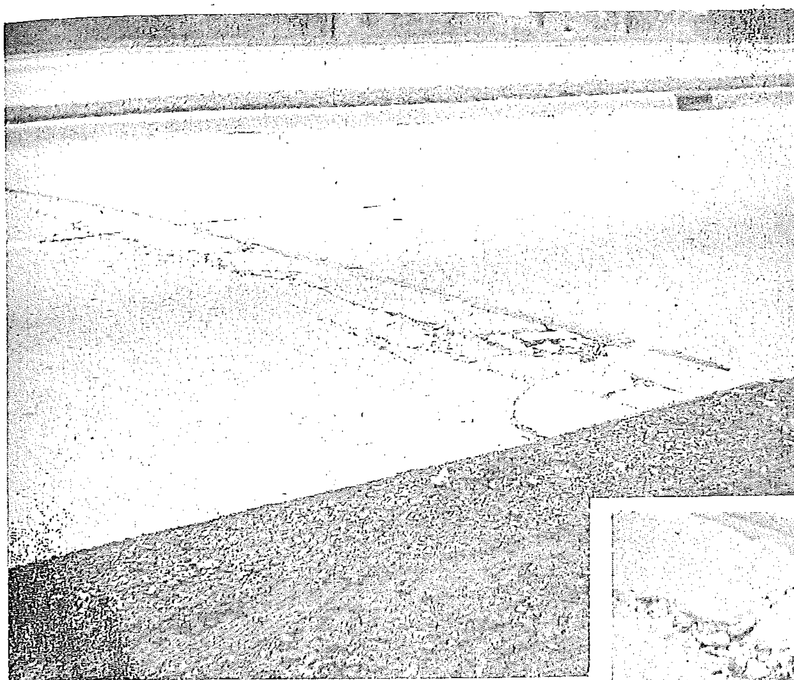
Over the years since 1946 several changes were made in the design and material of baseplates. In the 1950's rubber and plastic were approved as substitutes for galvanized steel but for various reasons were discarded as being unsatisfactory. Also, during this period, changes in base course construction 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.

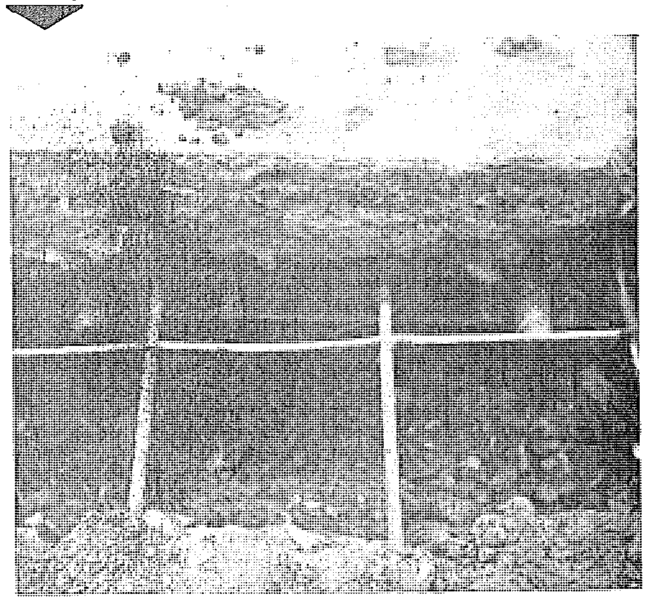
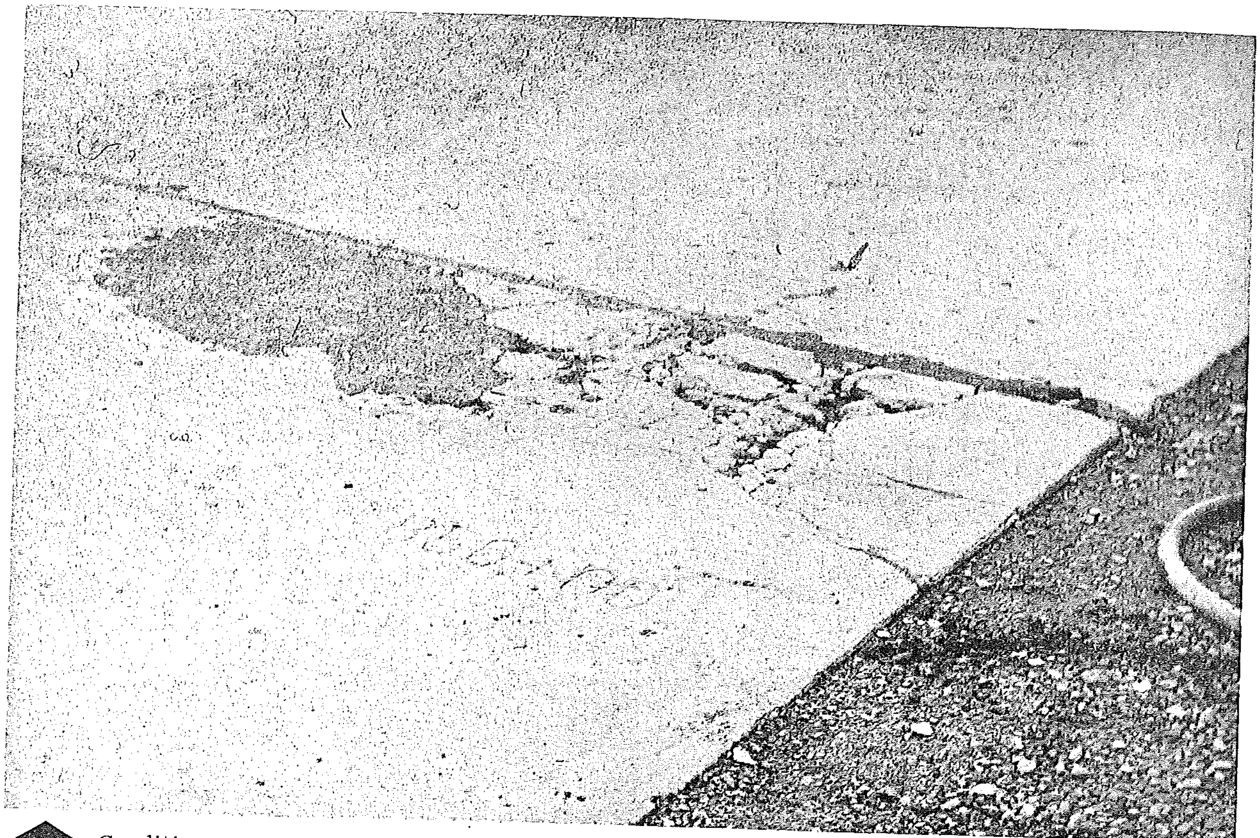
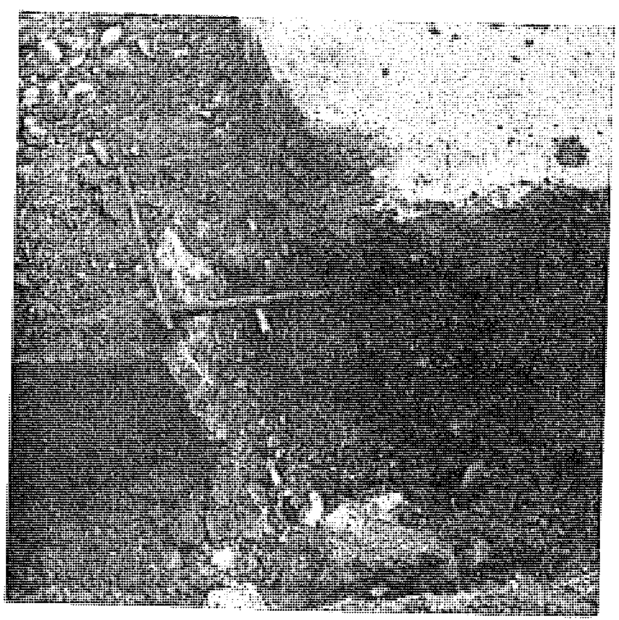


Figure 79. Condition and cause of joint problems.



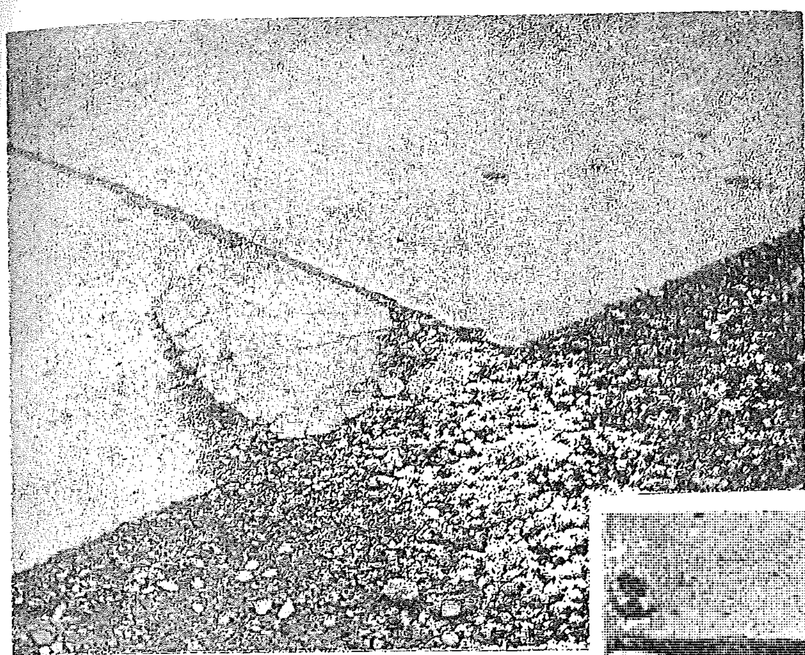
Condition of pavement joint on October 30, 1957.

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



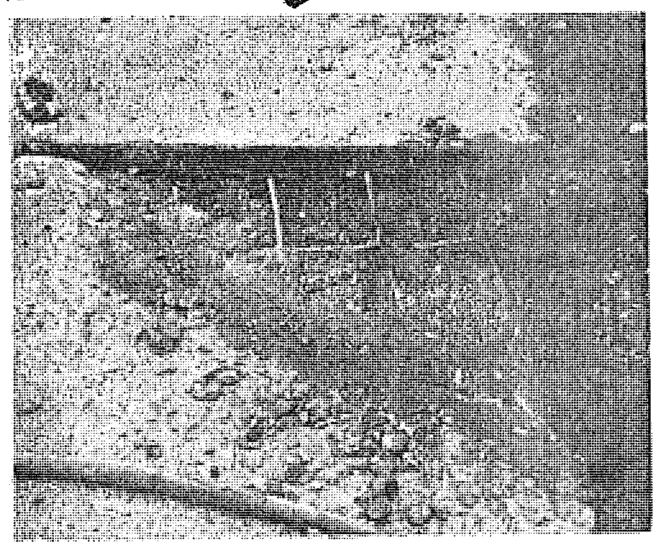
Edge dowel exposed showing 1-1/2 inches of lap across joint. Reinforcing steel passes through joint and although "necked down" was not broken.

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.

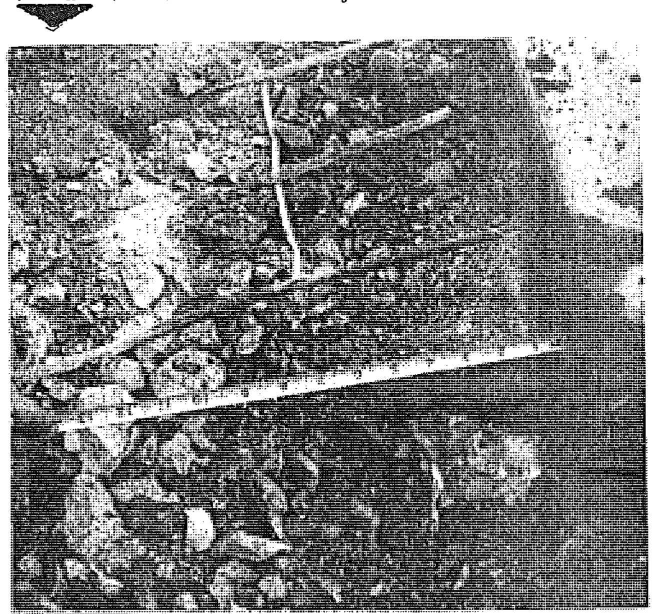


Figure 81. Condition and cause of joint problems.

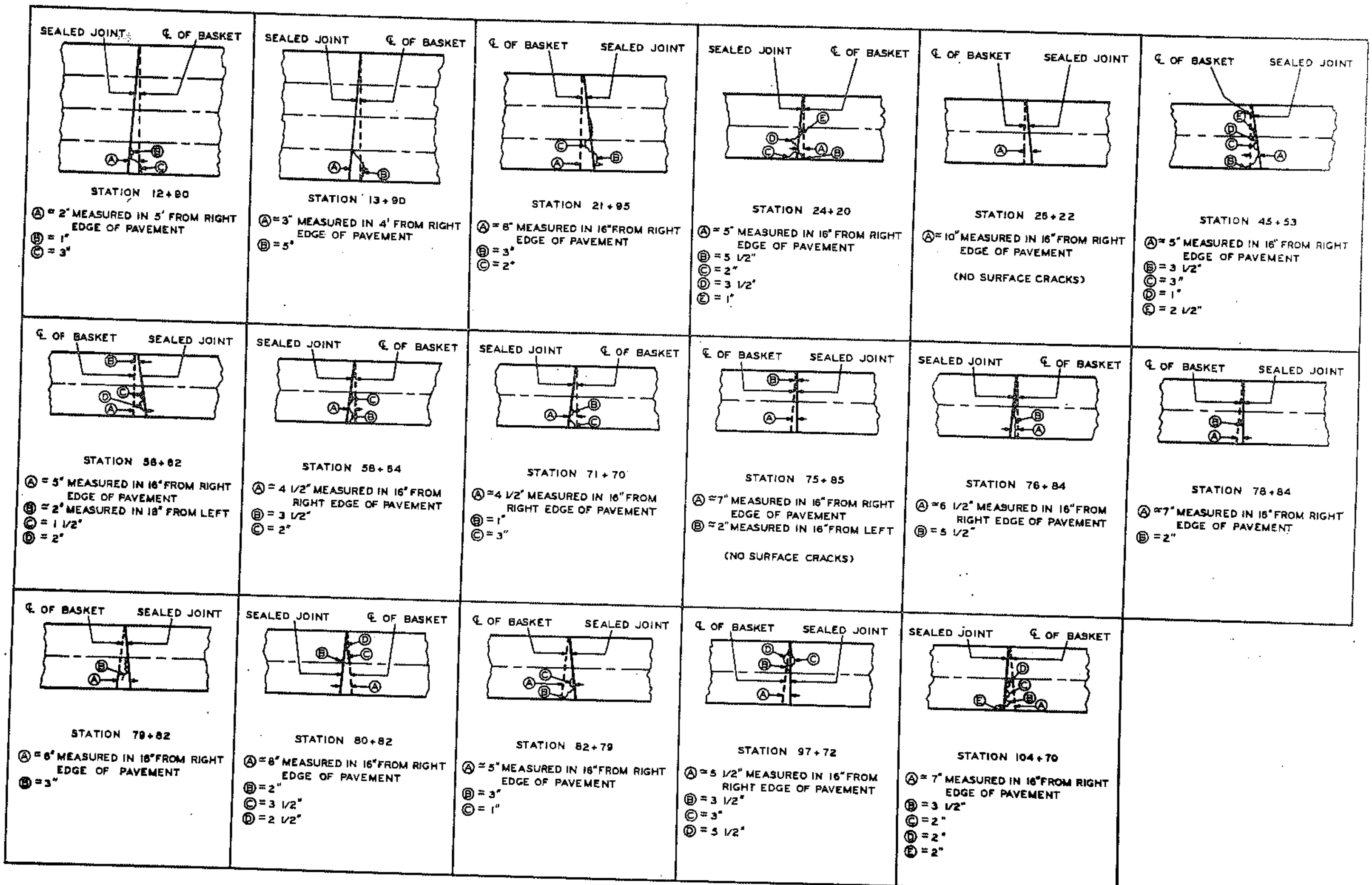


Figure 82. Condition and cause of cracking at various joints on project on US 2.

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

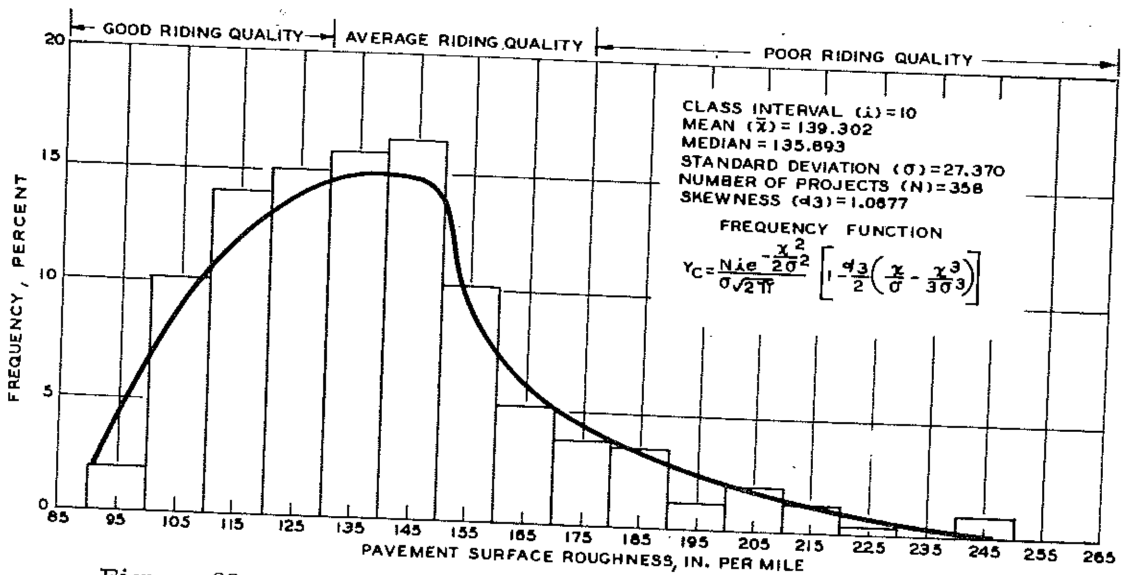


Figure 83. Frequency distribution of initial surface roughness values from rigid pavements constructed in Michigan; 1951-62.

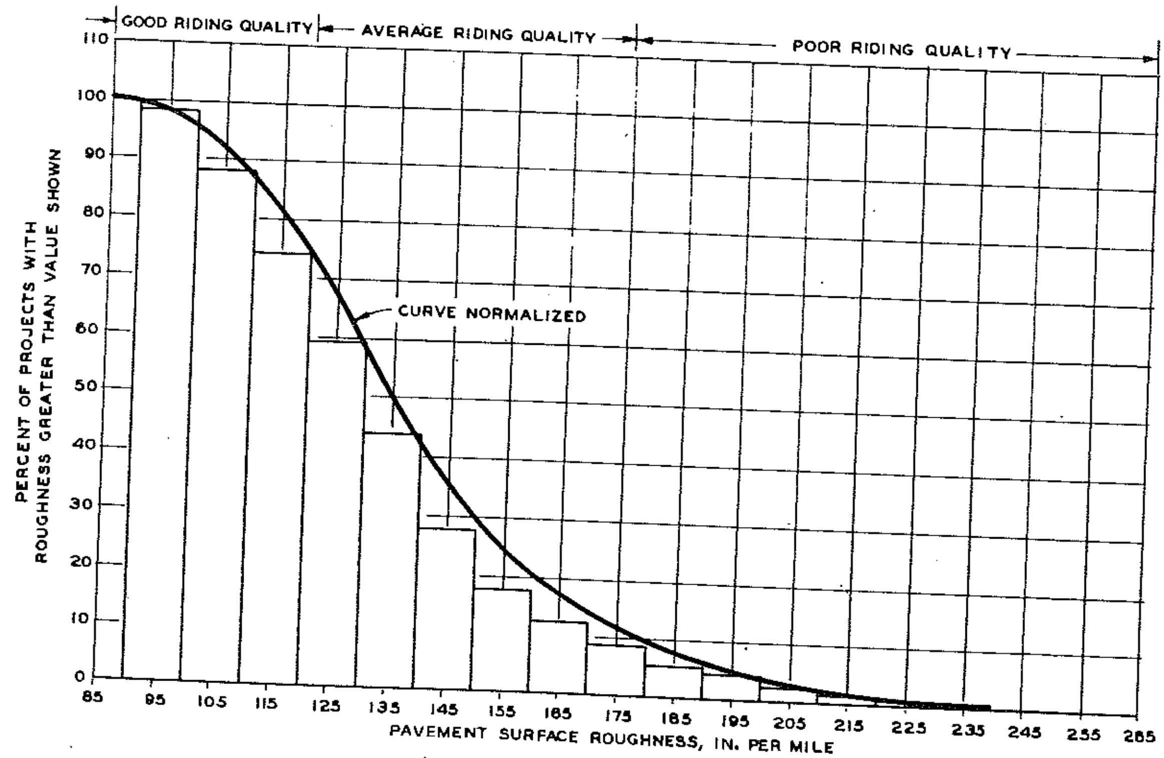


Figure 84. Cumulative frequency distribution of initial surface roughness values from rigid pavements constructed in Michigan; 1951-62.

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

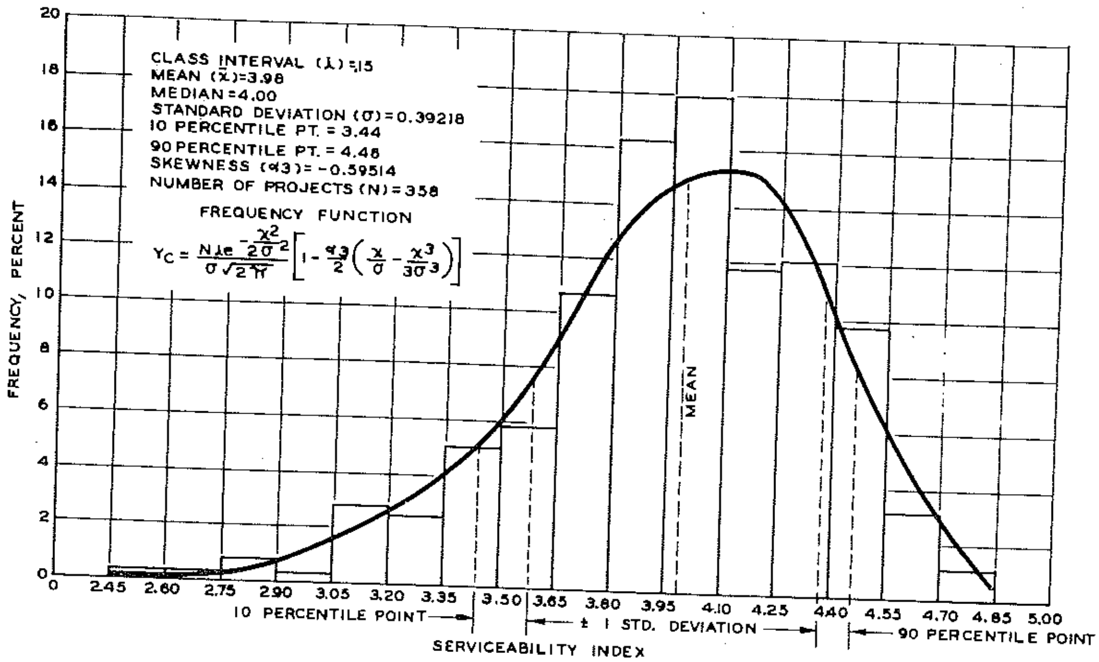


Figure 85. Frequency distribution of initial Serviceability Index values for new rigid pavement constructed in Michigan; 1951-62.

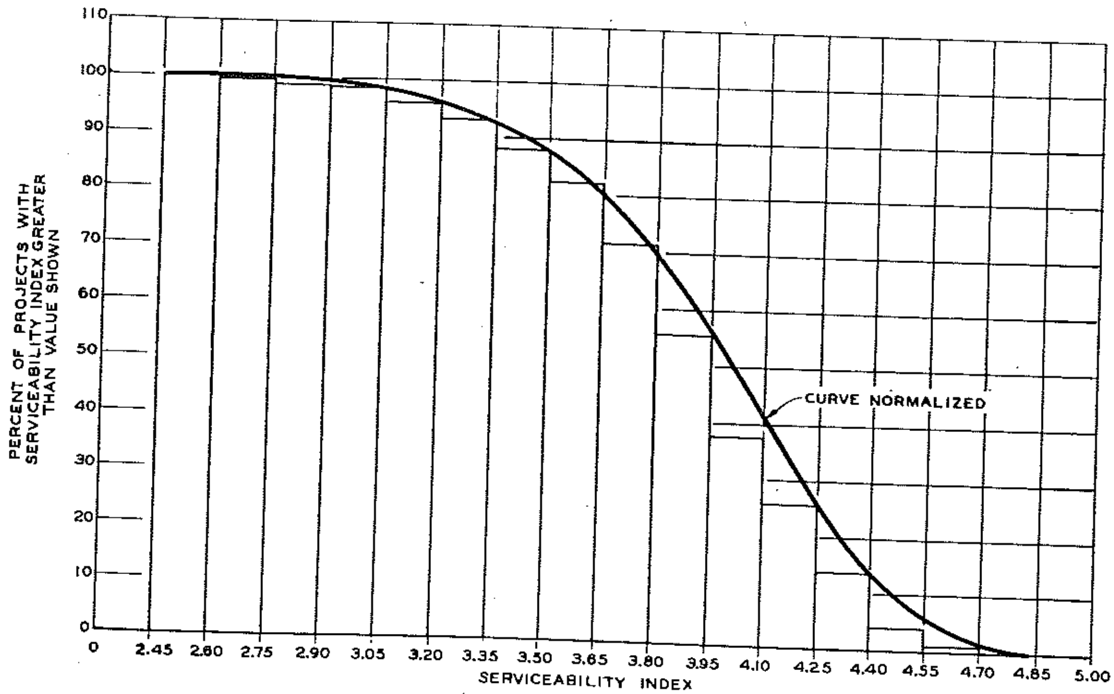


Figure 86. Cumulative frequency distribution of initial Serviceability Index values for new rigid pavement constructed in Michigan; 1951-62.

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.

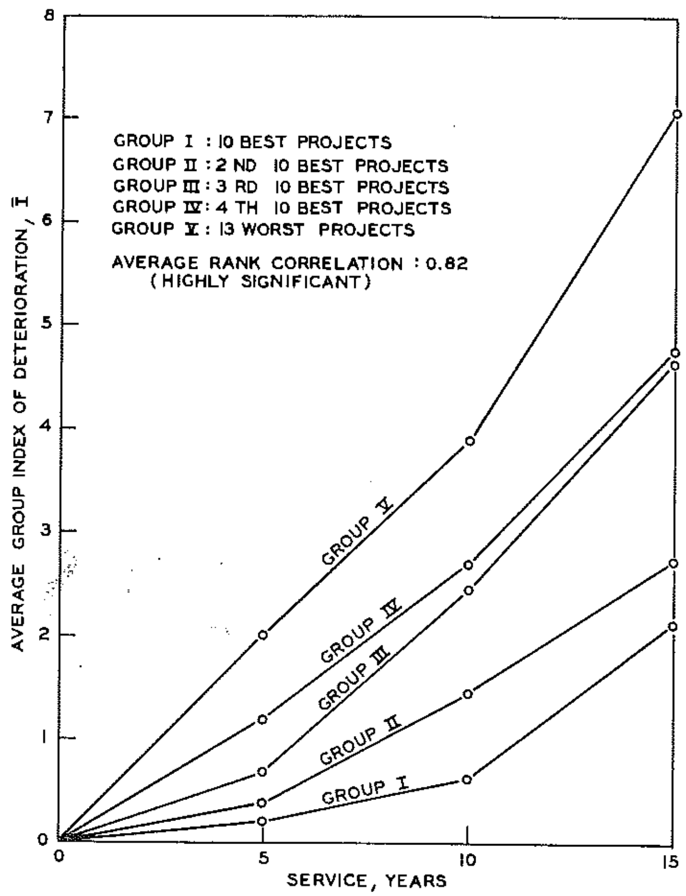


Figure 87. Five-year structural deterioration index as a predictor of future performance.

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

Cracks: 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.

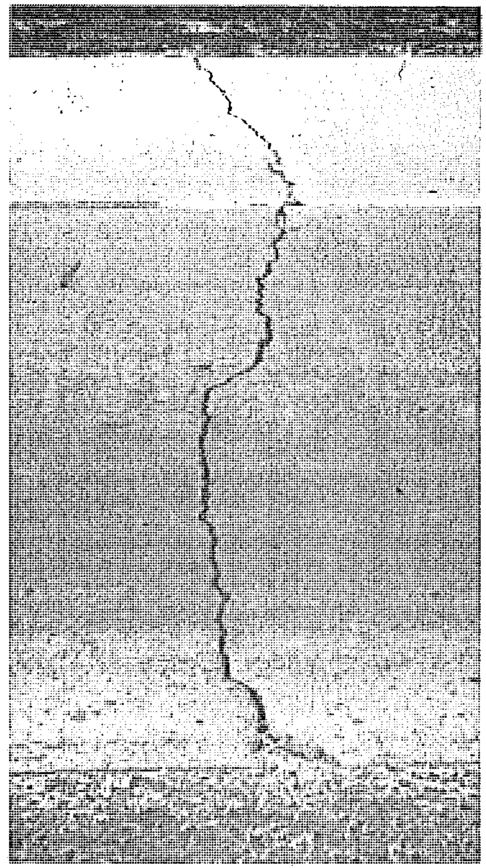


Figure A2. Typical diagonal crack.

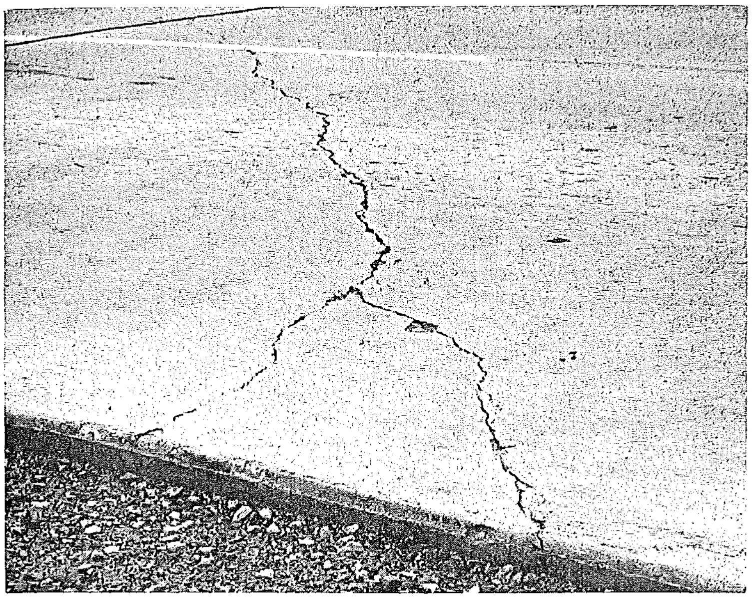


Figure A3. Longitudinal crack.

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."

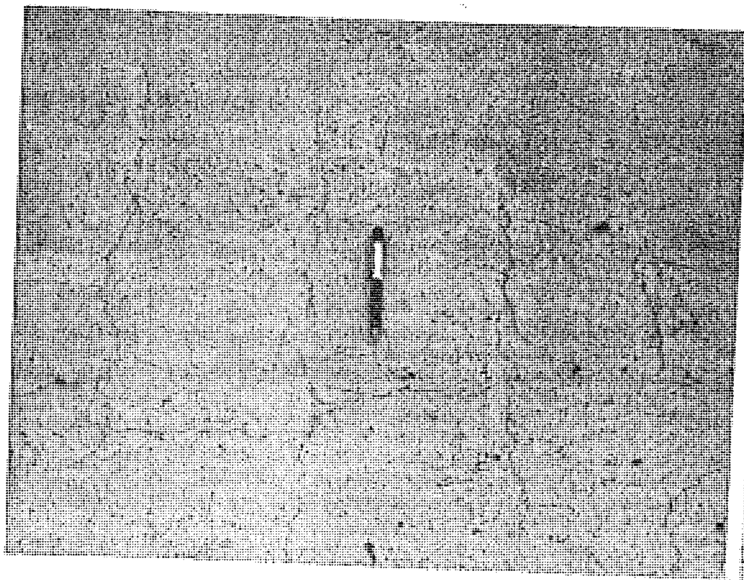


Figure A5. Map cracking. A form of disintegration in which surface cracking develops in a random pattern resembling political subdivisions on a map; may develop over the entire surface or only in localized areas; may or may not be associated with abnormal growth of the concrete.

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."

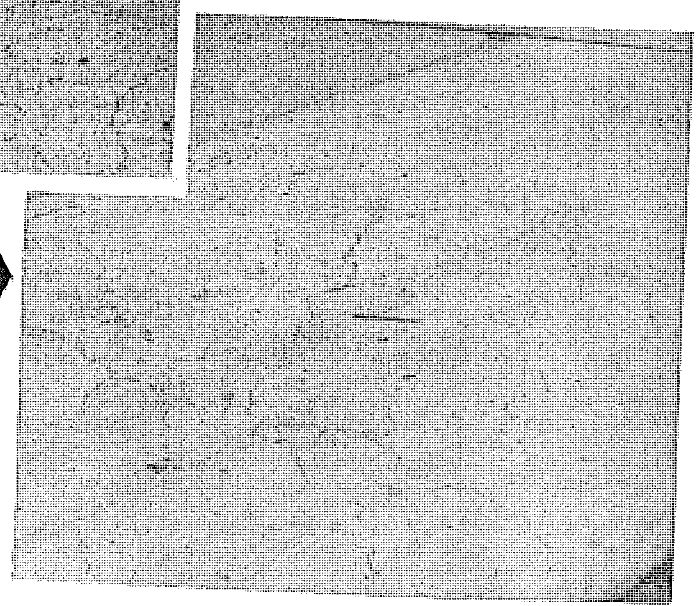


Figure A7. Faulting. The differential vertical displacement of the slabs adjacent to a joint or crack.

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

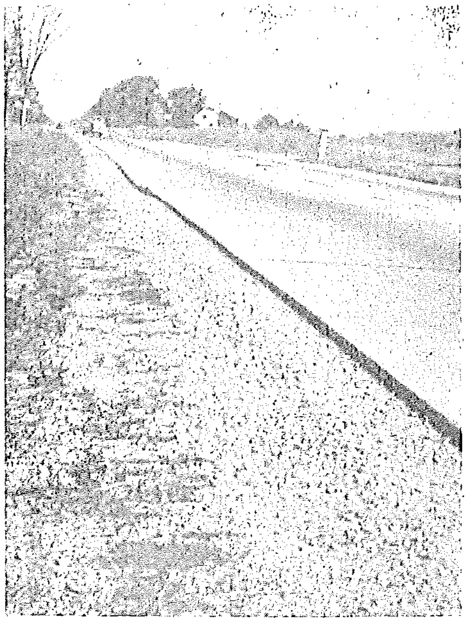
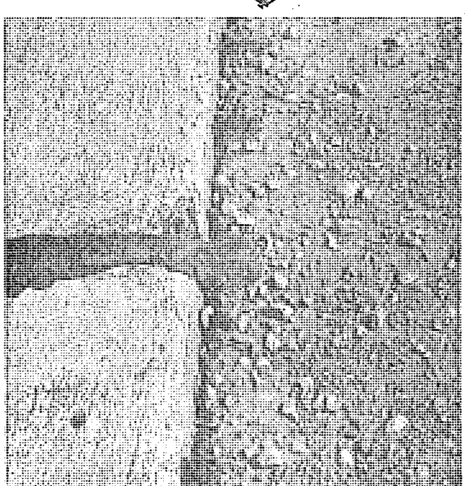


Figure A9. Settlement. Reduction in elevation of short sections of pavement or structures due to their own weight, to the loads imposed on them, or to shrinkage of the supporting soil.

Figure 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.



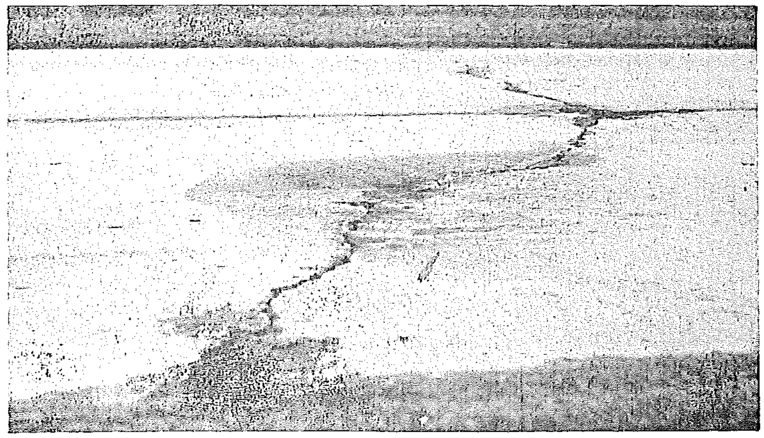


Figure A10. Pumping. The ejection of water or mixtures of water and clay or silt along transverse joints and cracks and along pavement edges caused by downward slab movement activated by the passage of heavy axles over the pavement, after the accumulation of free water in the subgrade or subbase. Sometimes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

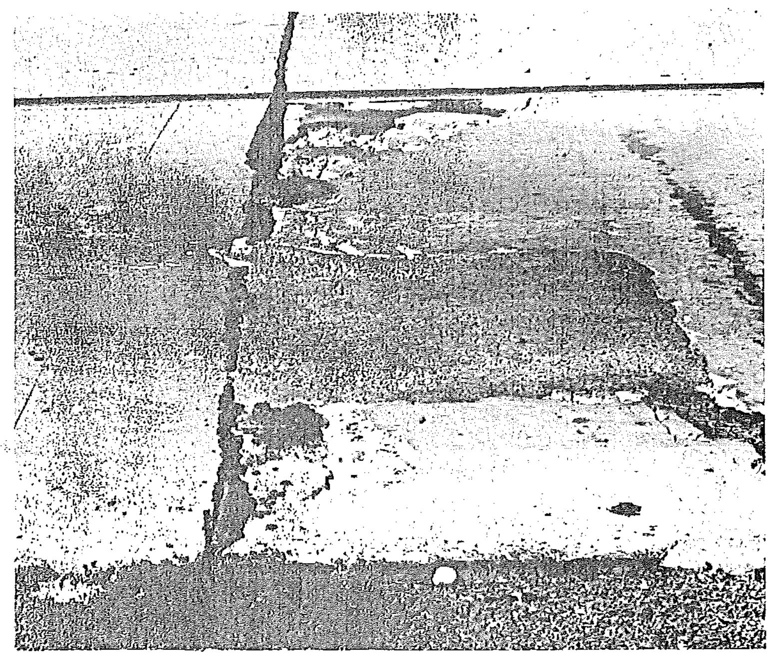


Figure A12. Patch. In an area less than a full lane in width, (a) the covering over with or without removal of the existing pavement, or (b) removal and replacement of all the existing pavement, with portland cement concrete, bituminous concrete, tar and chip mix, or other materials.

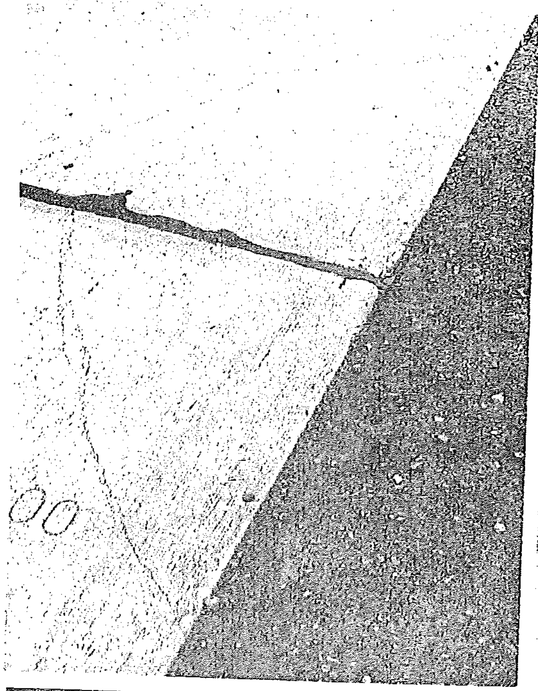


Figure A13. Corner break. A vertical fracture partially or completely through the slab, forming an approximate isosceles triangle with a transverse joint or crack and the outer or longitudinal joint; involves not more than half a lane width. Various types of corner breaks are as follows: corner break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

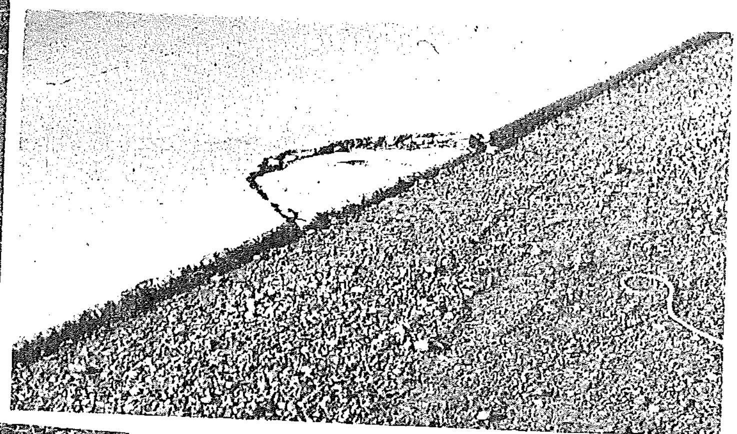
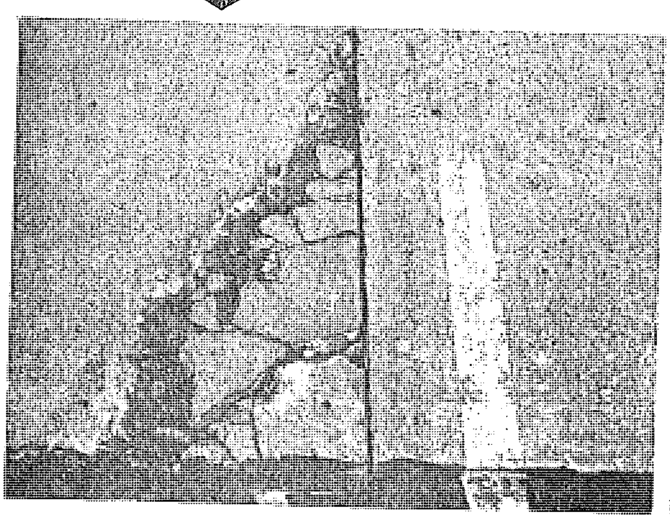


Figure A14. Edge break.

Figure A15. Slab corner spalling, exterior.

Figure A16. Slab corner spalling, interior.



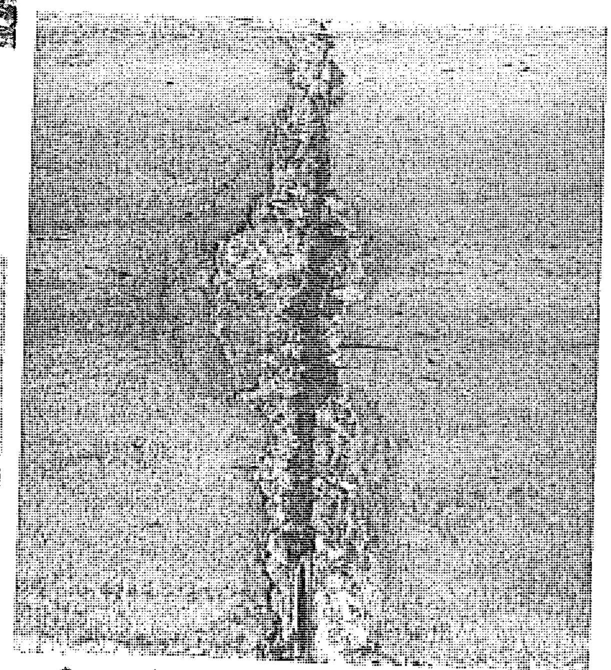


Figure A17. Joint spalling, interior.

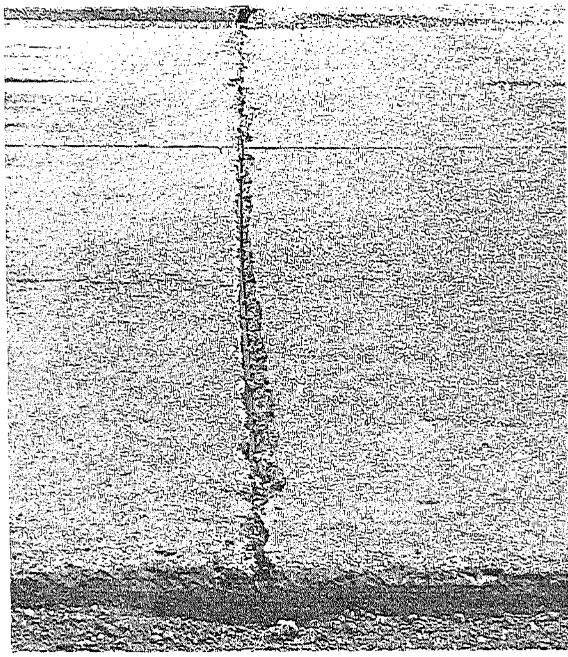
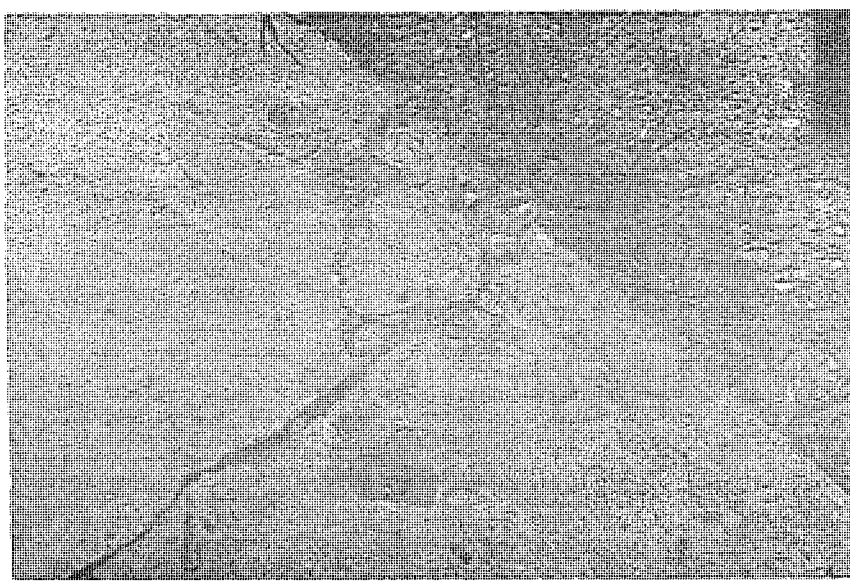


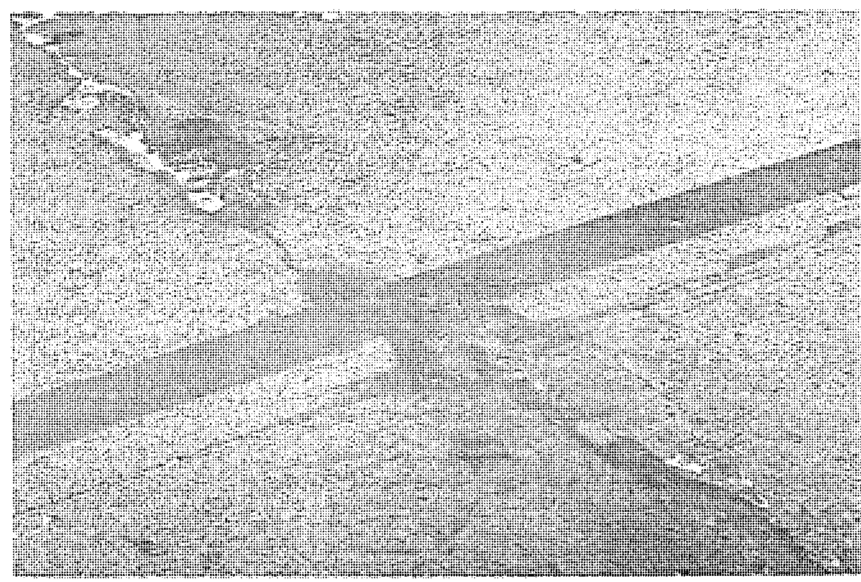
Figure A18. Joint groove spalling.

Figure A19. Outer edge spalling.

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

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





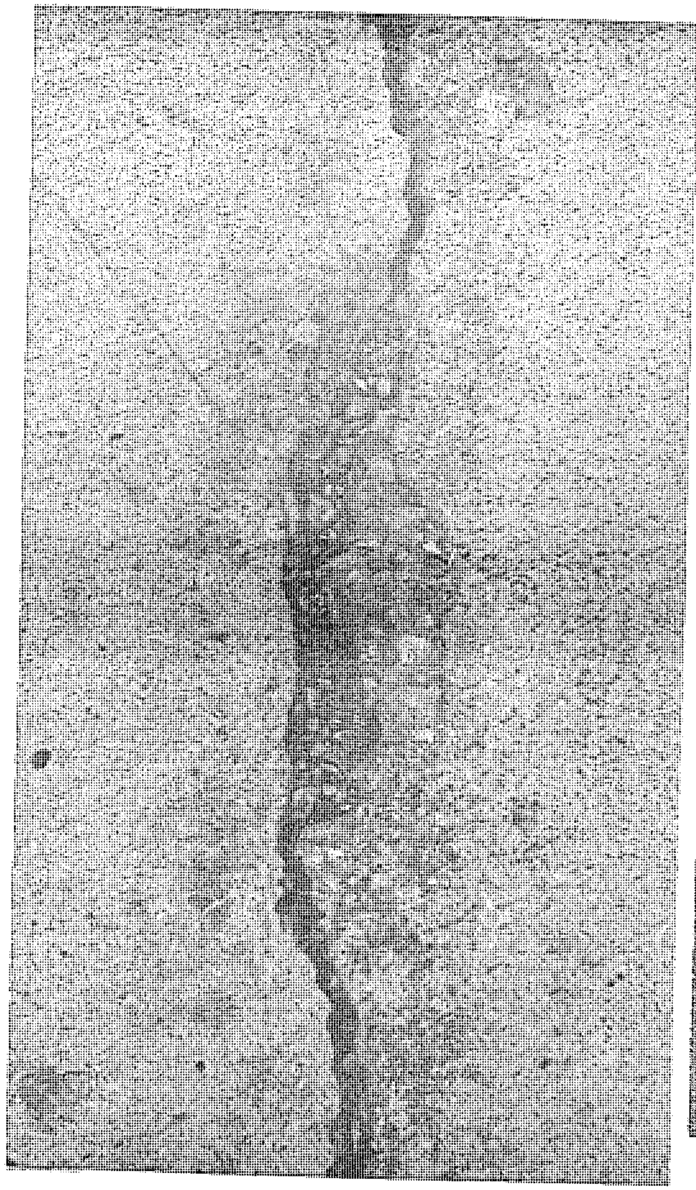


Figure A22. Spalling along crack.

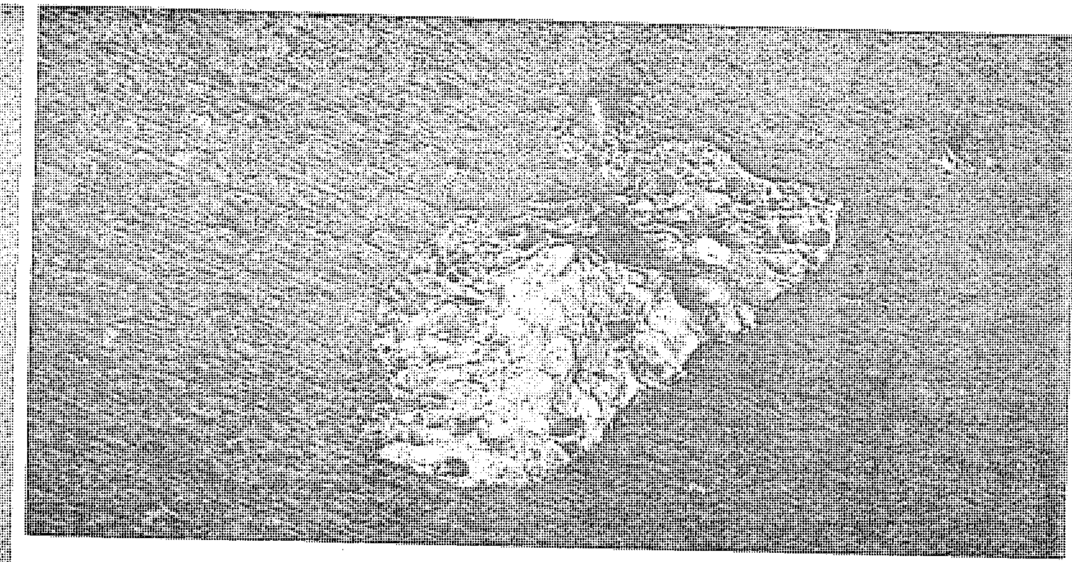


Figure A23. Surface spalling.

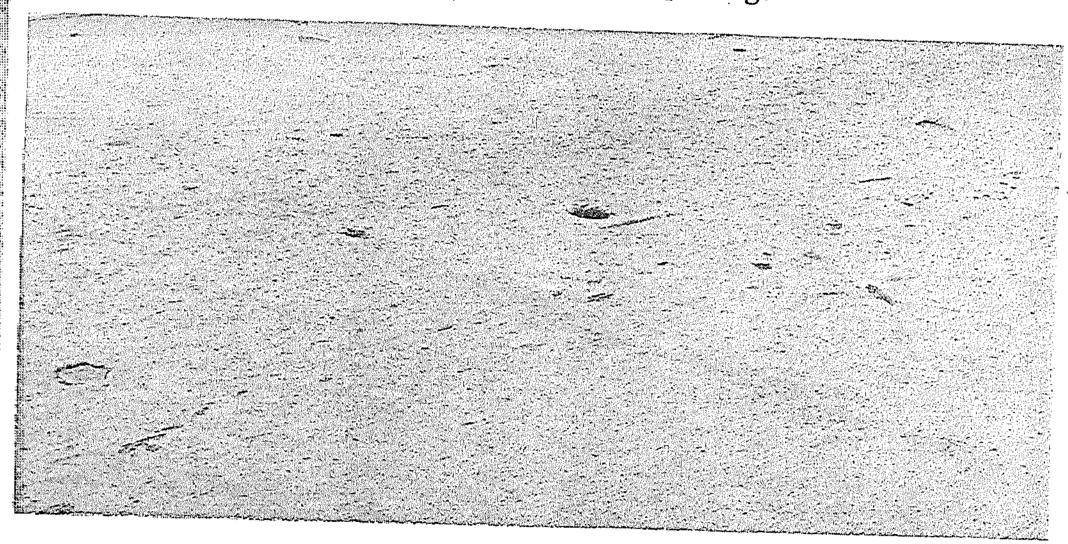


Figure A24. Mudjacking. A treatment to correct settlement of pavements. Holes are bored in the pavement and suitable materials pumped under the slab to raise it to the desired elevation.

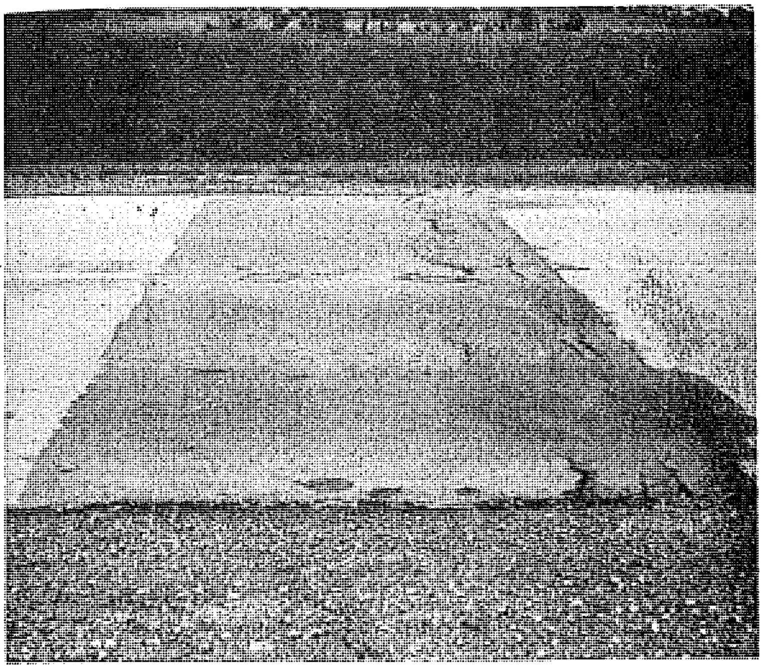


Figure A25. Resurfacing. In an area at least a full lane in width, (a) the covering-over with or without the removal of the existing pavement, or (b) removal and replacement of all existing pavement with portland cement concrete, bituminous concrete, tar and chip mix, or other materials.

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

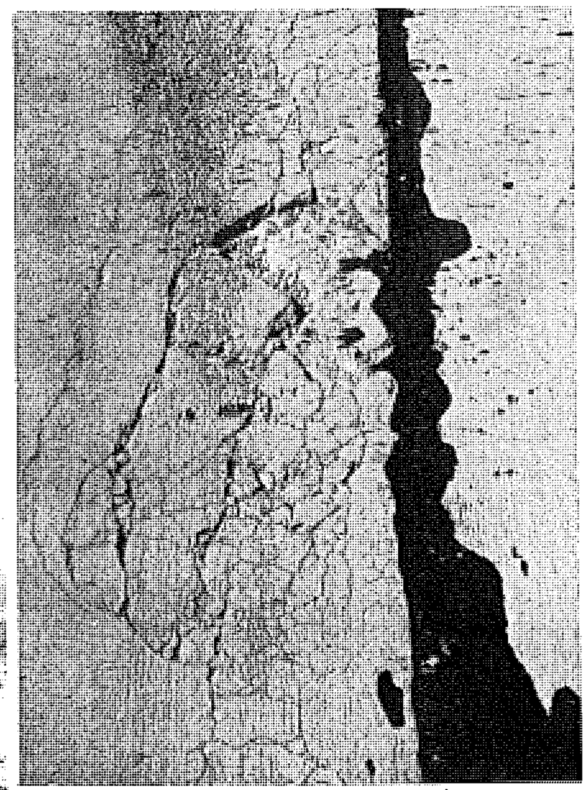


Figure A27. Frost heave. The differential upward displacement of pavement due to action of frost which has caused localized swelling of the subgrade.

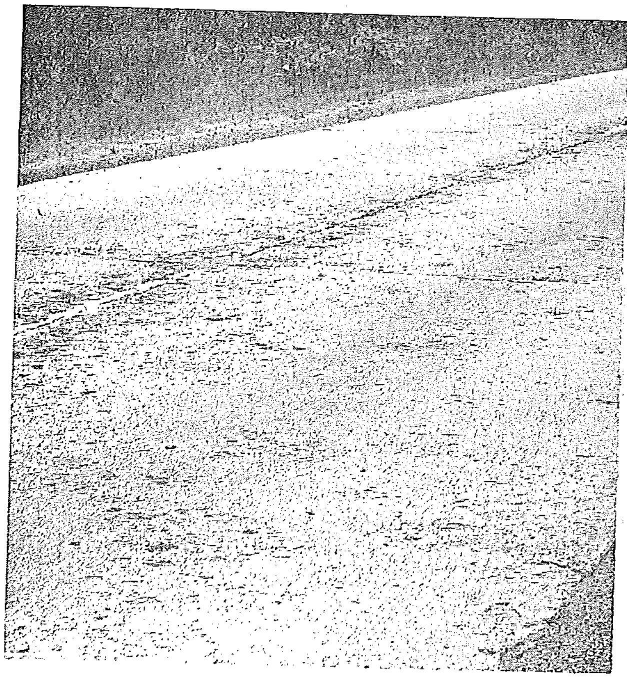


Figure A28. Surface scale. The peeling away of surface mortar exposing sound concrete, even though the scale extends into the mortar surrounding the coarse aggregate.

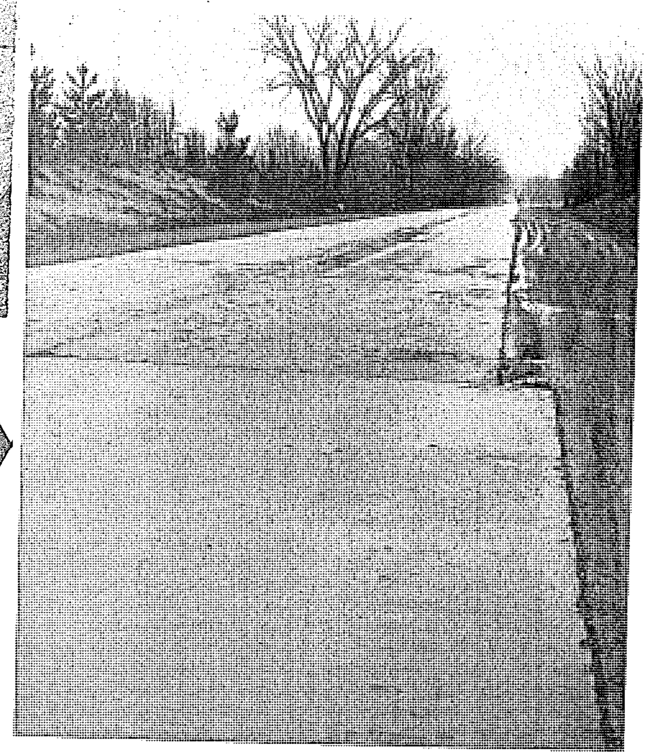


Figure A29. Progressive scale. A condition of concrete disintegration which in its initial stages appears as surface scale, but gradually progresses deeper below the surface stratum. Tapping or drawing a hammer over such areas generally produces a hollow or "plunky" sound.

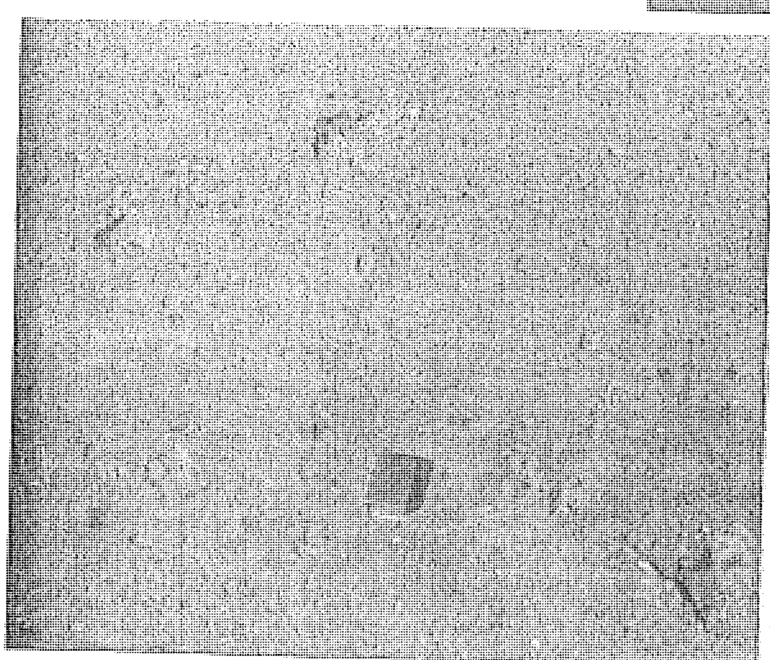


Figure A30. Pitting. The displacement of individual aggregate particles from the pavement surface, due to the action of traffic or disintegration of the particles, without major displacement of the cementing material or mortar.

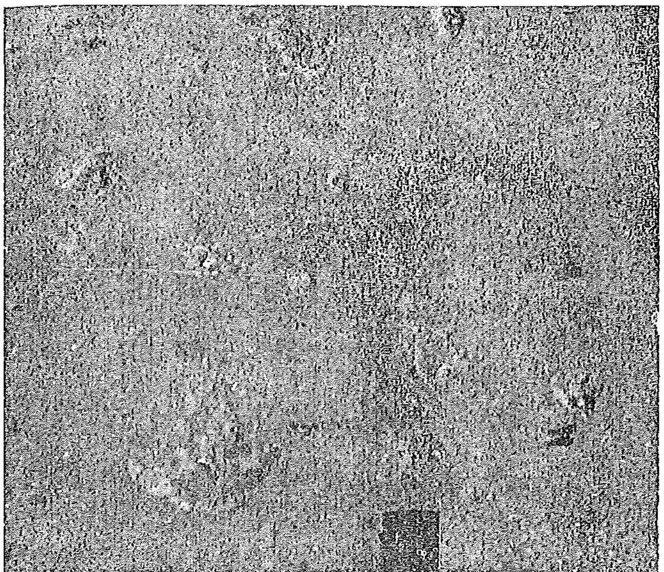
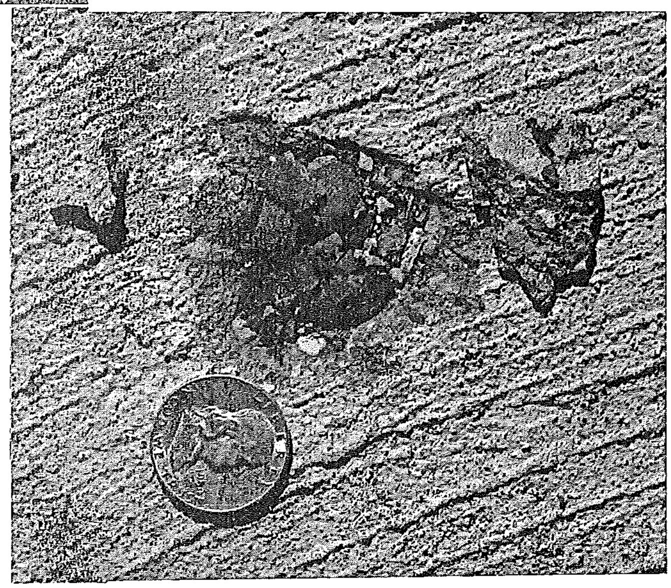


Figure A 31. Flecking. Dislodging of the thin mortar film over coarse aggregate particles near the pavement surface, resulting in exposure of the particles; generally attributed to a lack of bond between the mortar and aggregate. This is generally an early stage of pitting and although limited in extent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



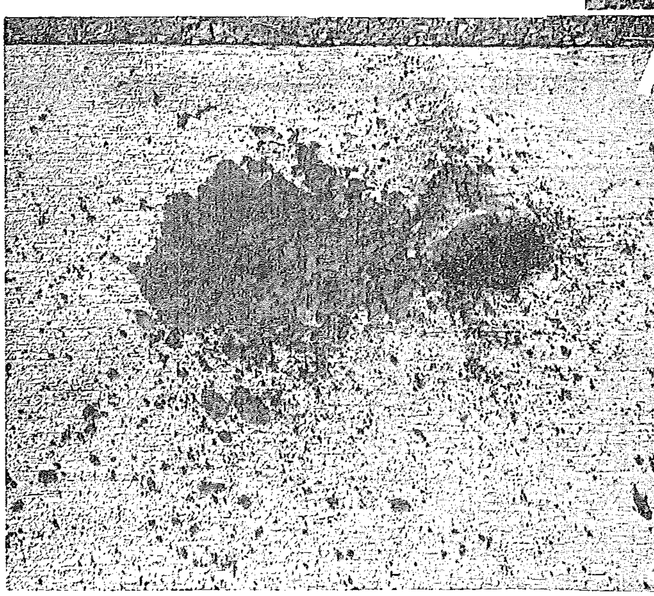


Figure A33. Clay pocket. A hole in the pavement resembling a pop-out, caused by a lump of clay in the aggregate used in construction; the result of disintegration under freeze-thaw conditions and traffic.

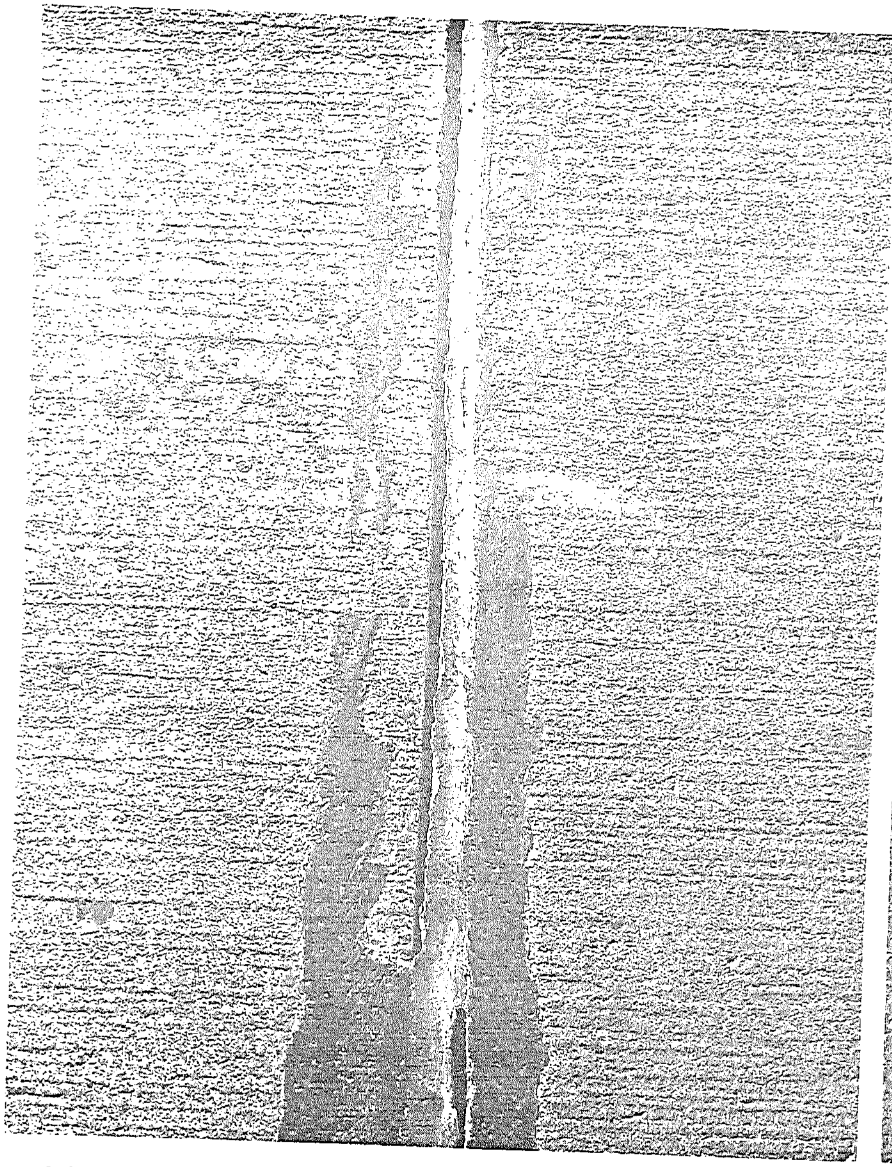


Figure A34. Joint seal adhesion failure. A bond failure between joint seal and the face of the joint seal groove.

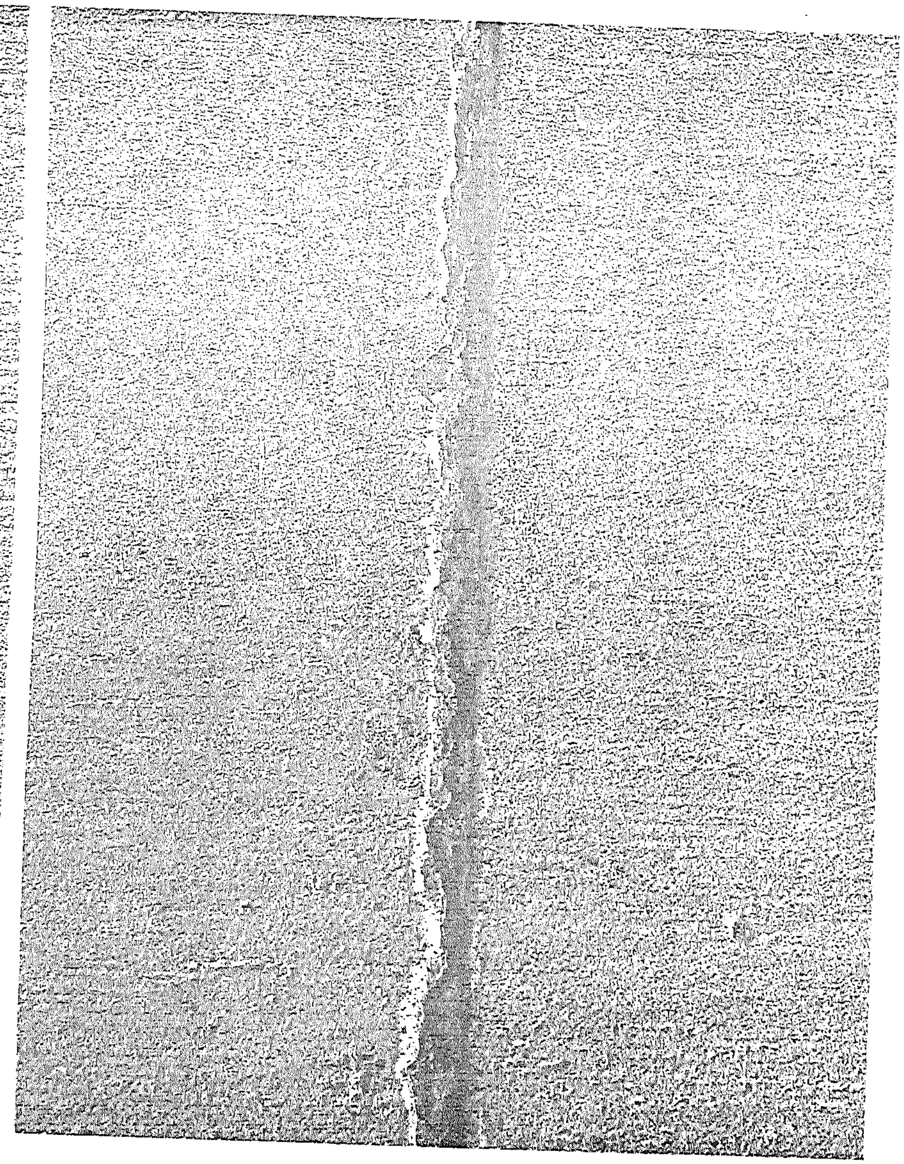


Figure A35. Joint seal cohesion failure. A bond failure within the joint seal itself.

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.