

RECOMMENDATIONS CONCERNING CONCRETE
PAVEMENT DESIGN AND MAINTENANCE

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MICHIGAN DEPARTMENT OF STATE HIGHWAYS

**RECOMMENDATIONS CONCERNING CONCRETE
PAVEMENT DESIGN AND MAINTENANCE**

**Research Laboratory Section
Testing and Research Division
Research Project 39 F-7(14)
Research Report No. R-790R**

**Michigan State Highway Commission
Charles H. Hewitt, Chairman; Louis A. Fisher, Vice-Chairman
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Lansing, November 1971**

This report has been prepared in response to questions presented by the Pavement Selection Committee at the meeting of September 1, 1971. The minutes of the meeting list the following specific questions:

- 1) What is recommended regarding expansion joint spacing?
- 2) What is our experience with the Acme joint assembly?
- 3) What is recommended regarding reducing the joint spacing to 60 ft or less?
- 4) What is recommended regarding a preventative maintenance program, which--by contract--could eliminate the majority of our blow-up problems?

Conclusions and Recommendations are presented here in the same order as the questions noted above, with details and reasoning supplied in the remainder of the report.

1) It appears that the use of expansion joints in normal "warm weather" pavements, is not of sufficient magnitude that the elimination of such joints at many of the traditional locations would significantly affect the long term performance of the pavements. Candidates for elimination are the following:

- a) Beginning and ending points of long-radius curves
- b) "Spring-points" of crossroad intersections.

Expansion space should be maintained:

- a) Adjacent to structures and railroad crossings
- b) In the intersecting roadway at "T" configurations
- c) Adjacent to sharp curvatures, such as in ramps (Note the I 96-Cedar St. ramp problem)
- d) At the nose of ramps.

Adoption of this policy may require the installation of additional expansion space at structures, and the incorporation of expansion space at repairs as noted below.

2. Experience with the Acme joint has been good to date. The state of New York has used them for many years. A portion of an Acme device was removed from the Oakland Avenue pavement on November 1, 1971 after 7 years of service. The assembly did not show serious deterioration due to corrosion. Acme devices and plastic coated dowels are recommended as replacements for our presently specified steel dowels, on a project-by-project basis.

3. Shortening of slabs by one mat length, to 57 ft-3 in. , might have some merit if present seals are retained. We would not expect a very marked change in performance from this action, but some improvement in sealing probably would result. We do recommend, however, that improvements in load transfer devices be made to incorporate corrosion resistant or plastic coated dowels, or Acme type devices, regardless of whether slab length is changed.

4. We recommend a preventative maintenance procedure, based on selection of the poorer joints for replacement prior to blow-ups. Sealed joints and expansion space should be provided at each repair (unless two adjacent joints are repaired, in which case the expansion space probably could be eliminated from one of them).

Background Information

Serious deterioration of jointed pavements begins at the joints. In general, approximately 90 percent of the pavement area is still in good condition at the time that repairs are required for localized areas of deterioration, usually at the joints. For this reason, there have been many efforts to reduce the number of joints in concrete pavements. The ultimate in this respect is the continuously reinforced design. However, the longer the individual slabs become, the greater the amount of movement that takes place at the joints. This presents serious problems in keeping the joints properly sealed. Even the best poured-in-place sealers cannot be depended upon to seal joints in slabs greater than about 17 to 20 ft in length, over a long period of time, if joint grooves are kept at traditionally acceptable limits of width.

In Michigan's existing pavements with 99-ft slabs, the hot-poured seals were effective only during the first year or two of life. Infiltration of salt, entrapped by the baseplates, resulted in deterioration of the joint faces, proceeding from the bottom upward. Incompressible materials also enter the joints. Twelve to fifteen years after construction the joint faces are destroyed to the point that they can no longer support the compressive forces caused by moisture and hot weather, and total joint failures begin.

The Research Laboratory issued reports beginning in 1957 concerning the deleterious effects of base plates and inadequate joint seals. The results of these observations are summarized in MDSH Research Report R-582 (1966). In 1962 the first preformed neoprene joint seals were installed experimentally on I 96 South of Lansing. Also in 1962, the I 96 joint test road was constructed. Evaluation of this project, along with the advent of pre-

formed joint seals, resulted in the adoption of our present design. Slab length was reduced from 99 ft to 71 ft-2 in., base plates were eliminated, and preformed neoprene seals in sawed joint grooves were required. These changes were instituted in the Design Division from 1964 to 1966, and were implemented during the construction season of 1967. Some projects already under contract were altered by authorization during the interim period. Therefore, it is now possible to compare the condition of pavement joints constructed in different manners, although the age of the pavements involved is not yet sufficient to develop serious problems of deterioration. Cores were taken recently from joints on I 96 near Lansing, I 196 in Grand Rapids, and I 69 near Marshall. These limited results indicate that neoprene seals provide additional protection for joints, even when base plates are present, and the removal of base plates has provided further improvement. There is still evidence of some infiltration of fluids into the joints, but incompressible materials are effectively excluded. There still is evidence of the initiation of deterioration of the joints, but it obviously is proceeding at a rate that is much slower than that of the previous design.

Concrete pavements are designed for a 20 year life, but we have shown no tendency to tear them up after that life has been reached. Many are still in use far past the design age, and with one or more resurfacing treatments, 40 to 45 year old pavements are still carrying traffic. However, joint problems in the original pavement are not cured by resurfacing. They crop up continuously, erupting through the blacktop, causing roughness and serious maintenance problems for years to come. Joint faces should be sound if the overlay is to be durable.

Expansion Joints

There has been considerable discussion of the merits of expansion joints. Most states install expansion joints only adjacent to structures. About six states require the use of expansion joints at prescribed intervals and/or temperature conditions, but in no case is their use very extensive. Experiments in one state showed that expansion joints spaced at about 30 ft gave good performance over a 25 year period, while other sections with expansion joints spaced at 40 ft or more, deteriorated at a much faster rate. Another state found that on certain pavements, joint blow-ups per mile of pavement were fewer in areas between closely spaced structures, where greater expansion capability exists. There have also been instances where blow-ups occurred at the joint adjacent to a newly installed expansion-relief joint. Studies of joint deterioration and blow-ups usually indicate interaction of several factors, with no apparently consistent relationships that can be determined from the uncontrolled variables.

There is little doubt that the use of many expansion joints would reduce or delay the occurrence of blow-ups ("many" in this case means expansion at nearly every joint). However, expansion joints are more expensive and far more difficult to install properly. In fact, it is practically impossible to consistently install good expansion joints with slipform pavers. A poorly installed expansion joint means problems at an early age, which in turn requires maintenance. Since the expansion space is not required until the pavement is quite old, some additional early maintenance problems would be built into the road in order to prevent or delay maintenance problems at a later date.

Michigan's rural pavements built during warm weather, have a relatively small proportion of expansion space installed, and that appears to be used up during the first few years of pavement life. Performance records of "cold-weather" pavements, with expansion space at 396-ft intervals, indicate that these pavements perform about equally with those placed at higher temperatures with fewer expansion joints. Evidently, the amount of space provided no more than makes up for the additional expansion due to differences in placement temperature.

Load Transfer Devices

Ideally, a load transfer device must accomplish two things; first, it must transfer loads across pavement joints without faulting and second, must have the ability to allow unrestrained longitudinal movement of the pavement slabs. There are many methods and numerous devices for use in transferring loads across joints. One simple method, and by far the most common, is the use of round steel bars installed across the joint. This method is quite satisfactory from the standpoint of load transfer efficiency, but with respect to joint movement restraint, considerable improvement is desirable.

A brief description of the cause and effect of movement restraint in joints, where load transfer is provided by use of steel dowels, follows:

Prior to installing the bars a debonding agent is applied to the sliding portion of the dowel, and on dowels for expansion joints a cap to provide expansion space is also put on the sliding end of the dowel. Once the concrete is poured and the plane-of-weakness sawed, the shrinking and temperature stresses cause the first movement to occur. Depending on the weather condition at time of pour this initial opening of the joints takes place anywhere from one to several days after pour. Generally speaking, joint movements during the first few years are uniform from joint to joint. However, after four to five years some joints begin to restrain movement more than

others and unequal joint openings result. Also, tensile forces increase in slabs adjacent to the restrained joints.

The increased restraint of joints appears to be caused by rusting of the steel dowels. Rusting of dowels is accelerated by leaky seals and disintegration of the bond coat on the dowels. The moisture (and in the winter, salt brine) enters through the seals and seeps along the dowel where the breakdown of the debonding agent has left a vacant space. The rust scale forming on the bars effectively increases their size and induces high pressure on the concrete surrounding the bars. This pressure results in a higher force being required to move the dowels.

Joint restraint, as previously mentioned, results in unequal joint movements and increased tensile forces in slabs adjacent to a restrained joint. Consequently, joints with the larger openings may deteriorate faster because of the ease by which detrimental liquids can enter. The increased tensile forces cause transverse crack openings near mid-slab that accelerate rusting and eventually the reinforcement fractures. It should be noted that totally restrained joints and cracks with fractured steel are not as much of a maintenance problem as deteriorated joints. However, their elimination would reduce and defer joint maintenance.

An example of the effect of joint restraint is afforded by construction project BI 41024, C2 RN on I 96 in Kent County, constructed in 1959. Of a total of 914 slabs, 327 (36 percent) had an open crack (open crack defined as 1/16 in. or more in width) when surveyed in 1969. These cracks had occurred between five and ten years after construction. A recent cursory inspection revealed that numerous cracks had fractured steel and that joint problems are prevalent.

A good deal of work has been done to determine the force required to pull out a restrained dowel or dowels that have been in service a number of years. For example, Van Breeman (1) reports that pull-out loads on 12 channel-type dowels embedded for six years averaged 20,000 lb per dowel. Perkins (2) found that a 1 in. diam by 18-in. long steel dowel, in one case, required a force of 4,000 lb to pull out, and in another case 8,000 lb was required to pull out a dowel of the same type. These dowels were in service three years prior to testing. Mitchell (3) reports that on the basis of data from his pull-out tests the restraining force per dowel varies from 3,000 to 8,000 lb. The dowels tested had been embedded from five to eight years. Six 1-1/4 in. by 18-in. long dowels in service for eight years were reported by Haviland (4) to have a pull-out resistance ranging from 7,200 to 13,600 lb. Pull-out tests conducted by the Research Laboratory on samples of dowels removed from various test installations ranging in age from 6 to 12 years showed a pull-out resistance of 800 to 9,600 lb.

Until recently, the only way to prevent dowel corrosion has been to use stainless steel dowels. Now a plastic coated dowel has been developed by Republic Steel Corporation and several test installations have been made. A cast malleable iron load transfer device (Acme assembly) has been in use in New York state for several years and the material itself is less corrosive than our presently used steel dowels.

Experimental installations of both the Acme assembly and plastic coated dowels have been made by the Department. Six joints containing Acme assemblies were constructed on Oakland Avenue in Lansing in 1964, and 71 joints on M 52 near Owosso were constructed in 1969, with this type of assembly. The Owosso project also included 10 joints containing plastic coated dowels.

The Acme assemblies used on the Oakland Avenue project were designed for 60-ft slabs and were installed primarily for evaluation of construction feasibility of the Acme joint system in use at that time. The addition of a left turn lane at Capital Avenue resulted in four of the joints being covered with an asphalt overlay. Observation of the remaining two joints indicate satisfactory load transfer and movement performance. A portion of an Acme device was removed from the Oakland Avenue pavement on November 1, 1971. The assembly did not show serious deterioration due to corrosion.

The Acme assemblies on the Owosso test installation were re-designed to accommodate the movements of the 71 ft-2 in. slabs. The joint grooves were sawed and sealed in accordance with our specifications. To date, joint width variations have been quite uniform and load transfer tests conducted in December 1969 showed that the average load transfer effectiveness of both the standard assembly and the Acme assembly was 90 percent. The load transfer effectiveness is defined as the ratio of the deflections of the unloaded side of the joint to the loaded side. There is no appreciable difference in the transverse crack pattern or number of cracks in the Acme and control sections. The initial joint spalling survey indicated a total of 3 in. per joint in the Acme section compared to 3.7 in. in the control section. The surface roughness was 10 inches per mile higher in the Acme section than in the control section, but both sections were in the range of "average" roughness.

The ten joints containing plastic coated dowels have performed equally as well as the joints with Acme assemblies, except their load transfer effectiveness was 75 percent compared to the Acme's and standard dowels 90 percent. However, it should be pointed out that because of the relatively small deflections measured, a very small amount of difference in deflec-

tion from the unloaded to loaded side, when given in percent, will show what appears to be poor load transfer effectiveness. For example, the 15 percent difference between the standard and plastic coated dowels actually results from the unloaded side of the assembly with plastic coated dowels, deflecting on the average only 0.002 in. more than the unloaded side of the standard assembly. Plastic coated dowels removed from an Ohio installation five years old indicated the coating was satisfactorily preventing corrosion of the dowel.

The list price of the Acme assembly was 32 cents more per lineal foot than the standard assembly when the installation was made two years ago. Estimated cost increase in assemblies containing plastic coated dowels appears to range from 25 to 50 cents a lineal foot, which in current joint spacings amounts to about 3 to 6 cents per sq yd of pavement.

On the basis of the information available to date it appears that both the Acme assembly and the plastic coated dowels offer less resistance to joint movement than our present steel dowels, with little or no sacrifice in load transfer capability. It is, therefore, recommended that these two load transfer devices be specified for the present on a project-by-project basis.

Slab-Length as Related to Initial Cost and Deterioration

There has been a great deal of discussion during the past few years concerning the reduction of slab length in jointed pavements. Naturally, shorter slabs would cause less joint movement, which is sometimes thought to improve the possibilities of proper joint sealing. Shorter slabs also have been touted as an improvement in riding quality. This seems to have no basis in fact, and may result in exactly the opposite effect if slabs are shortened sufficiently. Roughness of short slabs is affected to a greater extent by warping, and by a tendency for easier alignment with minor differential settlements. Some have blamed long slabs for joint deterioration but this develops due to lack of adequate seals. Indiana, with slab lengths of about 40 ft, has about the same troubles with deterioration that we do with our 99-ft slabs. The State of Illinois formerly used 100-ft slabs, with experience much like our own. That state has elected to use continuously reinforced design for most of their paving jobs since 1966.

Generally speaking, it seems reasonable to use joint spacings as long as possible, consistent with the ability to seal the joints. Apparently our neoprene seals are doing a fair job of sealing joints spaced at 71 ft-2 in. Shortening the slab by one mat-length, to 57 ft-3 in. would reduce joint movements somewhat, and perhaps would make the seals more effective, if the same size seals were used. Slightly smaller seals theoretically could do

the job with very minor savings in seal cost, but this would place the seal design for the shorter slab length at approximately the same proportion of compression as presently used with the larger seals and longer slabs, so little or no improvement in sealing efficiency would be accomplished by such a move. Since the cost of the seal itself is a minor portion of the joint cost, such action would not be recommended.

In general, our present costs for jointed pavements breaks down approximately as follows: concrete 79 percent, steel 12 percent, and joints 9 percent, when the cost of both expansion and contraction joints is included. Shortening the slabs would have very little effect on the cost of joints per foot, and therefore would increase the total cost by the amount required to install more joints. There would be a slight reduction in the proportion of expansion joints, if expansion joint spacings and usage policy remained unchanged. Under these circumstances, the proportional costs for 57 ft-3 in. slabs would be approximately 78 percent concrete, 12 percent steel, and 10 percent joints; and in 43 ft-4 in. slabs, the respective costs would be about 76 percent, 11 percent, and 13 percent. These increases, along with other minor changes that should be made, bring the cost of jointed pavements ever closer to the cost of continuously reinforced pavements, and it becomes questionable whether jointed pavements should be built. However, we should note here that the problem of mat translation encountered in slip-forming jointed pavements could be disastrous if it occurred in continuous pavements, since the maintenance of adequate lap in the mat splice is an absolute necessity in continuously reinforced pavements.

Therefore it appears that some improvement in joint sealing might occur if the slabs were shortened to 57 ft-3 in. and the same seals maintained. There would be a proportional increase in cost.

We are very strongly opposed, however, to the placement of very short slab pavements, or pavements without load transfer, on any main-line jobs where traffic volumes are significant or where speeds exceed 40 mph. Also, since traffic volumes on even rural roads have a history of increasing far more than anticipated, we suggest that in no case should load transfer be eliminated from main-line pavements. A previous report on County Road 151 and Square Lake Rd. (MDSH Research Report R-743) and others, have discussed this matter in detail, and it will not be carried further here.

Blow-Ups and Joint Crushing

Joint blow-ups have caused serious problems over the past several years. Summers with prolonged periods of hot weather provide conditions for the occurrence of more than the usual amount of such deterioration, especially

if the heat wave is preceded by heavy rain. The problem is not new, nor is it confined to the state of Michigan. A survey during 1966 revealed the following information. By July 26 of that year, Illinois reported an "...unusually large number of pavement failures due to hot weather." They reported that a large proportion of these failures occurred at joints, and further commented that "We call them all blow-ups. We have had 500 that have had permanent repairs and approximately 2,500 that have had temporary repairs." Indiana reported 834 emergency type and 7,073 non-emergency type joint failures during June and July, 1966. Ohio reported 245 "joint failures" during the same period. Wisconsin reported in October 1966, 429 "blow-ups" on portland cement surfaces during "... last summer's hot weather." They did not include blow-ups on rigid pavements that had been resurfaced. During that same summer, Michigan reported 1,882 blow-ups during June and July, 1,442 of them during June. Table 1 shows the total number of blow-ups reported by the Maintenance Division for the years 1966 and 1968 thru 1971. Figures for 1967 were not available.

TABLE 1
REPORTED BLOW-UPS

Year	Total No. of Reported Blow-ups
1966	1903
1968	1195
1969	851
1970	1030
1971	<u>1556</u>
Average	1307

It is apparent that the years 1968-70 were not as troublesome as 1966. However, extended hot weather during June of 1971 caused a series of blow-ups nearly comparable to June 1966. Also, in general, it can be expected that similar extended periods of hot weather in the future, during times when concrete pavement moisture content is high, probably will cause similar amounts of joint problems unless preventive measures are instituted. Yearly increases in traffic volumes make such occurrences ever more costly to the Department and irritating to the motoring public.

Since a large proportion of our concrete roadway mileage was placed during the late 1950's and early 1960's, using 99-foot slabs, poured joint seals and base plates, it is obvious that future requirements for maintenance of these pavements, will be very large.

Departmental records show that Michigan Interstate highways built between 1957 and 1964 contain more than 2,500 lane miles of concrete pavements with 99-ft slabs and poured seals, that contain nearly 136,000 such lane joints. Interstate pavements with 71 ft-2 in. slabs and neoprene seals built prior to 1971, total less than 600 lane miles with approximately 42,000 lane joints. As time progresses, a greater proportion of the existing pavement will move into the troublesome age group, and the proportion of joints that blow-up can be expected to increase as well. Therefore, if any progress is to be made, it is obvious that a significant commitment of funds will be an absolute necessity. If the funds are not committed to preventative repair, expenditures for emergency repairs will soar. Initial bids for contract replacement of joints by the precast slab method, have come in at \$40 per sq yd. If the average lane repair were 10 sq yd, this rate of repair cost would project to a potential total of over \$50,000,000 in the long-run to repair every one of the 136,000 lane joints indicated above, for 99-ft slab pavements.

It seems apparent that a maintenance program is required, that will make permanent repairs to the joints as they deteriorate, so that the quality of the pavement can be maintained over a longer period of time. If such a program were carried out during the life of the pavement, most joints would be of suitable quality to prevent crushing and blow-ups beneath a new wearing surface when it is applied. Therefore, the riding quality and length of service of the resurfaced pavement would be increased as well. It seems to be apparent also that repairs of this type should include expansion space and sealed joints.

Joint repair is an expensive proposition and, consequently, the selection of joints for replacement should be based on a reasonable probability that those joints would have crushed or blown-up within the next few years. In answer to questions as to whether joints can be selected prior to blow-ups, based on observable surface defects, the following information is of interest. Figure 1. shows the results of 10 and 15-year evaluations of 31.85 miles of two-lane pavement on 7 Michigan projects which were selected because of high incidence of blow-ups after 15-years. Observable defects present at 10-years of age were studied to determine the predictability of blow-ups within the next 5 years. Five areas were selected across the two-lane pavement joint, and the occurrence of observable defects within these areas on a particular joint at 10 years of age, was compared with the occurrence of a blow-up at that joint prior to the 15-year survey. The figure shows that 81 percent of the joints having defects in all five areas at 10 years of age, had blow-ups before 15 years of age. Conversely, only 3.1 percent of the joints that showed zero defects at 10 years had blown up by 15 years. It is

important to note here that in the preparation of the figure, no weighting was given to the severity of the deterioration that existed at 10 years of age. It was only noted that some observable defect, regardless of size, was present within the area. Therefore it seems obvious that by the application of a judgement factor, based on severity of existing deterioration at the time of inspection, it should be possible to further improve the accuracy of selection. Another interesting factor shown in the figure, is that if a joint were selected at random from the 7 projects, at 10 years of age, there was only a 14.1 percent probability that it would blow-up within the next 5 years. Thus, by considering the existence of observable deterioration in the various locations, the possibility of picking a joint that would blow-up within the next 5 years was improved by a factor of nearly six (81 percent compared to 14.1 percent). Generally speaking, the newer the pavement, the more idifficult the selection of the isolated locations where blow-ups might occur. The records show that during the period of about 8 to 12 years of age, joint failures are generally few and isolated in location, making them poor candidates for contract repairs. As the pavements pass the 12 to 15 year mark, joint problems become more prevalent, closer together, and lend themselves better to contract work.

Therefore it appears that two separate phases of concrete pavement maintenance are desirable depending on the age and/or condition of the pavement:

a) It would seem reasonable to establish a policy of making permanent repairs at the earlier, more isolated locations, utilizing MDSH Maintenance forces or contract county forces. Each joint, permanently repaired in this manner, will be one less bump in the road, one less location to plane or fill intermittently, and one less joint repair to contract at a later date.

b) Make permanent repairs by contract on older pavements.

Certainly, if we are to eliminate a large proportion of the blow-ups problem, and the associated traffic, safety, and financial problems that result, it will be necessary in the process to repair some joints that might have lasted another few years before ultimate failure. Also, it is not reasonable to expect that all future blow-ups would be eliminated, but a marked reduction should be possible. All possible effort should be made to keep both the former and the latter at a minimum, insofar as possible. Both will depend upon the accuracy of the selection scheme.

Before implementing a program of this nature as a standard maintenance procedure, we suggest the reliability of the proposed selection method be tested by an experimental program where the poorer joints would be selected by a process similar to that used for Figure 1, but with additional seve-

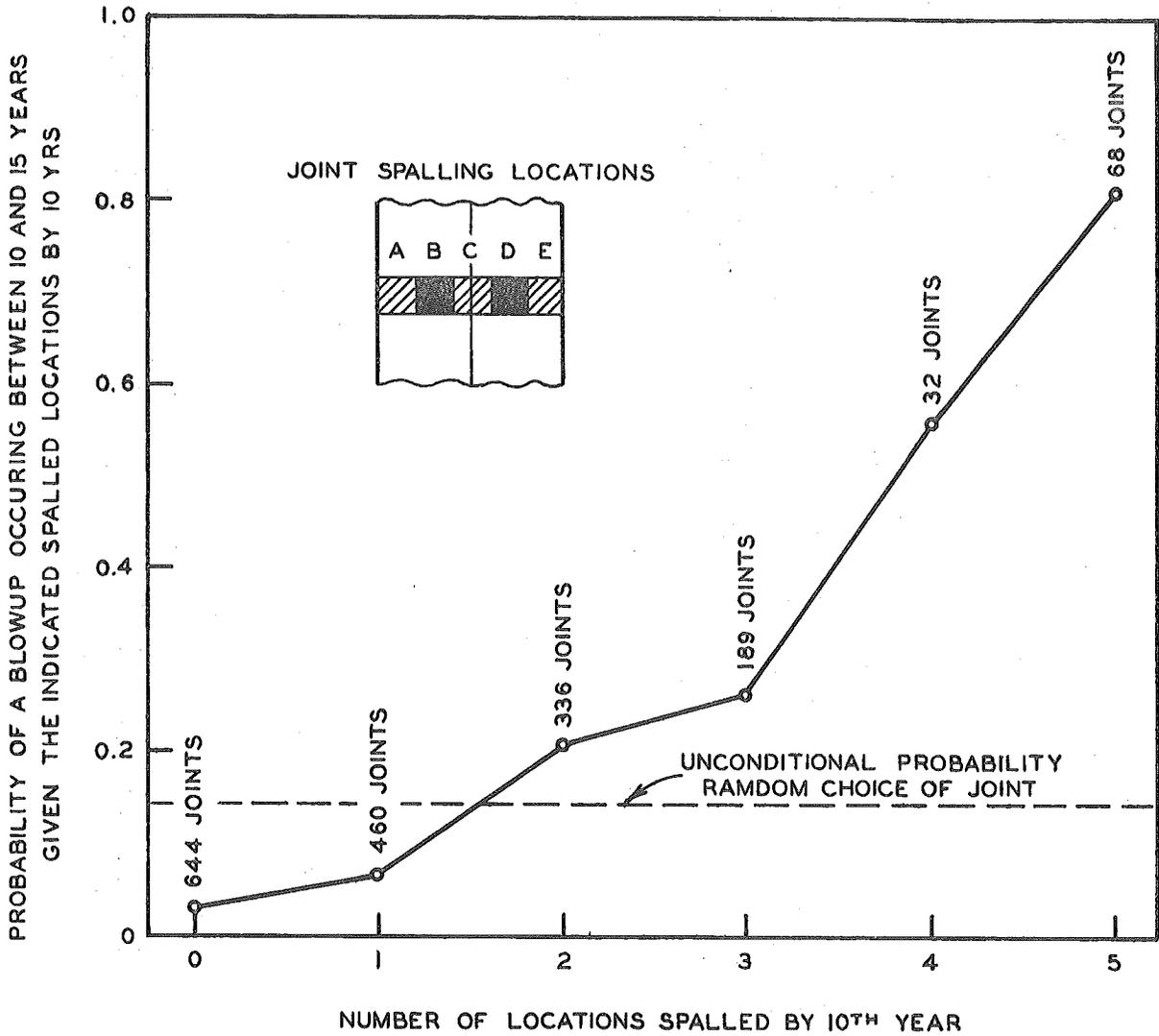


Figure 1. Predictability of blow-ups between 10 and 15 years on basis of joint spalling after 10 years of service.

rity factors added as previously noted. On one portion of the chosen projects, repairs would be made on the selected joints, either by contract or by Maintenance forces. On the remaining portion, operations would proceed in the normal fashion to determine the degree of accuracy attained in selection of the joints. A controlled experiment of this nature may provide information to more fully evaluate the merit of a preventive maintenance program.

Along with developing a program suitable for contract work our own maintenance forces would need to be oriented toward concrete pavement repairs suitable for present and future traffic values. In this respect our maintenance administration should be commended for their effort to develop speedy permanent types of repairs and obtaining the equipment capability to perform these types of repairs. Experimentation by Maintenance in cooperation with the Research Laboratory and contract counties, concerning full depth sawing, rapid removal methods, precast pavement slabs, incorporation of expansion space and joint sealing at repairs, has resulted in important improvements in procedures and results. New and larger concrete saws now in service, have increased capabilities to do this type of work when required, as well as to provide relief of pavement encroachment on structures.

Work with Genessee County on the precast slab experiment has indicated that permanent pavement repairs can be conducted as fill-in work during mild weather in fall, winter, and early spring. Construction of the precast slabs, indoors, provides similar work for the colder days of winter when snow removal operations have been completed. Therefore it appears that a policy of permanent concrete pavement repairs could serve a useful purpose in our relationships with contract counties, as well as provide the motoring public with better roads.

In summary the recommendations concerning preventive maintenance are as follows:

1. That MDSH Maintenance and contract counties take care of permanent repairs on pavements during the first 4 or 5 years after deterioration begins on a given project.
2. That MDSH Maintenance and Research personnel cooperate in the selection of joints on some older pavements where deteriorated joints are more numerous and more closely spaced, and that these joints be included in an experimental program for evaluation of the selection procedure and the preventive maintenance concept itself.

Available information indicates a probability of reasonable success with such a program.

Therefore, it appears that a preventive maintenance program to make permanent repairs, with provision for expansion space, should pay significant dividends in the reduction of emergency work and also be valuable in the long range use of the pavements as a base for new surfaces. The cost of such a program would be significant, especially during the first few years, in order to catch up on the backlog of temporarily repaired pavements in addition to the preventative work on a continuing basis as pavements deteriorate.

If sufficient funds can be obtained for such a program, there is little doubt of the benefits to be obtained from better riding surfaces over longer periods of time, as well as long-term benefits on resurfaced pavements.

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