The Problem of Corrosion of Load Transfer Dowels

ROBERT G. MITCHELL, Chief, Planning-Traffic-Design, Connecticut State Highway Department

The effect of corrosion on performance of load transfer dowels in transverse expansion joints in concrete pavements has been a subject of some concern for a number of years. This paper constitutes a report on the observations of dowel performance in transverse expansion joints on Connecticut pavements of an earlier design; that is, 100-ft slab length with three intermediate warping joints. It describes the condition of different types of hot-rolled dowels and beams removed from pavements of varying ages.

In a further attempt to evaluate various modern, rust-resistant coatings and claddings, standard abrasion and salt spray tests were performed at a commercial laboratory. Those materials with apparent promise were used in actual field installations and are under observation to determine their effectiveness in preventing seizure due to corrosion. These installations are in pavements of the more recently adopted 40-ft contraction joint design. Similar rust-resistant dowels were partially cast in concrete and submitted to accelerated test in a tidal area. The conditions pertaining in the various test installations in late fall 1959 are described.

●JOINTS in concrete pavement, and particularly certain troublesome features of joints, have been the subject of considerable investigation and study. The introduction of load transfer dowels helped to correct some earlier problems while creating new ones.

The effect of corrosion on the performance of load transfer dowels in transverse expansion joints in concrete pavements is not a new subject and has been of some concern for a number of years. An outstanding treatise on the subject is that by William Van Breemen "Experimental Dowel Installations in New Jersey," HRB Proc., Vol. 34, p. 8 (1955). Although less concerned with expansion joints, corrosion as an influence on dowel performance is also brought to attention in the paper by Ernest T. Perkins, "Test Project Constructed Utilizing the Contraction Joint Design," HRB Bull. 165, p. 35.

Further investigation by Connecticut of dowel corrosion resulted from observations made in July 1956 during an inspection of some pavements with Bengt. Friberg, Consulting Engineer. On the eastbound roadway of Route 15 in Manchester, a rather consistent, excessive opening of the transverse, weakened plane joint at the mid-point of the slabs was noted. These weakened plane joints are sometimes called "hinged" or "warping joints" as the pavement reinforcement is not interrupted at these joints. They will hereinafter be referred to only as "intermediate joints."

In each case, the joint seal at the expansion joints appeared to be in an undisturbed condition, whereas the seal in the open intermediate joints was quite obviously in very

poor condition. This seemed to indicate that slab movement, due to temperature changes, was occurring mainly at the intermediate joints, and that little or no movement was taking place at the expansion joints. Removal of a portion of the shoulder at one of the open intermediate joints revealed that the reinforcing steel through the joint was probably broken and the joint was now functioning as a contraction joint instead of as a tightly held crack. This pavement had been placed in 1948.

In September 1956, during an inspection of the concrete pavement of the eastbound roadway on Route 15 in the Town of Union, it was noted that several of the sawed intermediate joints were open from ½ in. to ¾ in. A general survey of this pavement, which was constructed in 1953, indicated a prevalence in the opening of the intermediate joints at the center of the slabs.

The pavement design at both of these locations was essentially the same. The expansion joint spacing was approximately 100 ft, with either formed or sawed intermediate joints spaced at approximately 25-ft intevals. Fabric reinforcement equal to 0.135 percent of the concrete area and weighing 61 lb per 100 sq ft was carried through the intermediate joints.

Random surveys, subsequently made, of concrete pavements constructed to similar design standards revealed the excessive opening of the intermediate joints to be quite prevalent. Because, in many cases, the joint seal in the expansion joints appeared in such exceptionally good condition, it was suspected that the load transfer dowels were either totally or partially restraining movement at the slab ends, with the result that compensating movement was taking place at the intermediate joints. In many instances, heavy trucks had caused the leading edge of the pavement at the intermediate joint to fault considerably, so that mudjacking was eventually required at locations of severe faulting.

PROCEDURE AND OBSERVATIONS

It was decided to cut out a portion of the concrete pavement containing one load transfer dowel at each of three expansion joints and a similar section of the concrete pavement across each of two wide open intermediate joints in the eastbound roadway at Route 15 in the Town of Manchester. A similar number of samples were also to be cut out of the eastbound roadway of Route 15 in the Town of Union. The purpose was to determine the condition of the dowels in the pavement, and whether the amount of restraint offered by the dowels to movement at the expansion joints might be the cause of the excessive opening of the intermediate joints.

Field

Figure 1 shows the location of the joints and the pavement areas cut out in the Towns of Manchester and Union. In October 1956, test blocks labeled 1M, 3M and 4M, were cut from the expansion joints and specimens labeled 2M and 5M from the open formed intermediate joints at the left edge of the inside lane of the eastbound roadway in the Town of Manchester. At the same time test blocks labeled 1U, 3U and 5U were cut from the expansion joints and two specimens labeled 2U and 4U from the sawed intermediate joints along the outside edge of the travel lane of the eastbound roadway in the Town of Union.

The use of a 22-in. concrete saw in cutting the 24- by 12-in. blocks from the 8-in. pavement resulted in some necessary over-cutting (Fig. 2). Areas cut out were backfilled with a cold bituminous mixture, but heavy truck traffic did cause cracking of pavement at the corners of pavement where over-cutting occurred.

Figure 3 shows expansion joints 1M, 3M and 4M on Route 15, Manchester, at which test blocks were cut out. The joint seal at all three joints is in very good condition with no indication of any adhesive or cohesive failures. A cross-section of expansion joint 4M showing the joint filler and the metal shield enclosing the filler as they appeared after 8 years of service is shown in Figure 4. The original ½-in. joint filler installed in this pavement in 1948 has been compressed to ⅛ in. The compressed condition of the joint filler is undoubtedly typical of a large number of expansion joints in concrete pavements of this design. Observations of ½-in. cork filler removed from the expansion joints on the Merritt Parkway in 1945 indicated a similar state of extreme compression along with frozen dowels.

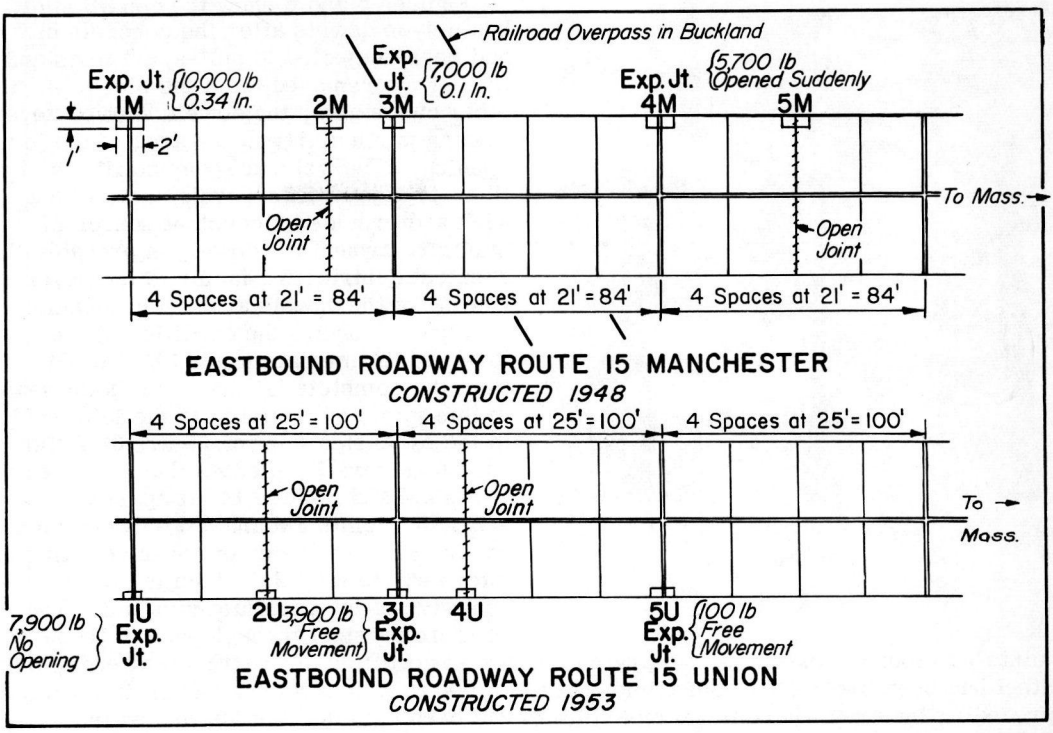

Figure 1. Location of joint cutouts.

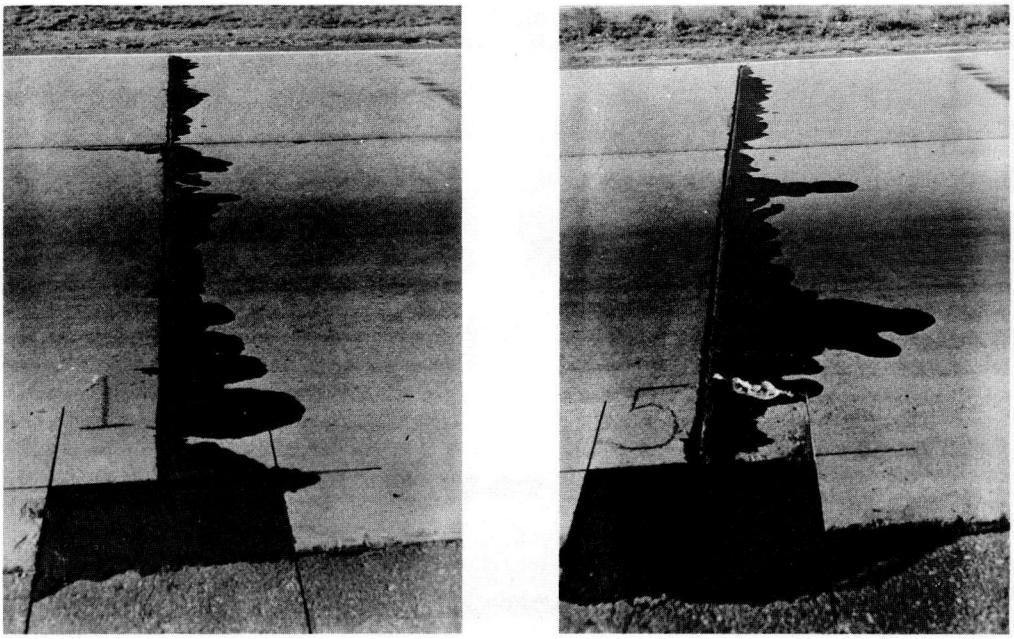

Figure 2. Expansion joints 1U and 5U on Rt. 15 in Union.

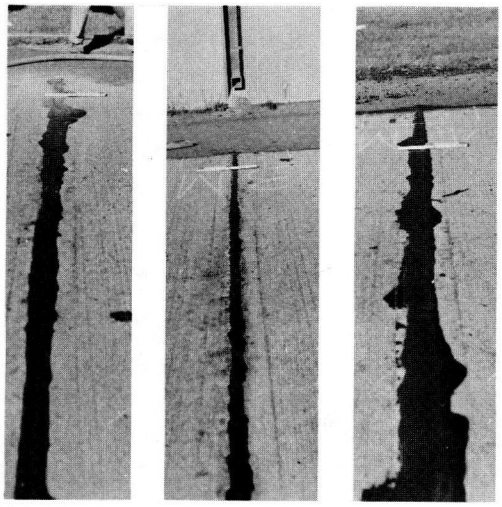

Figure 3. Expansion joints 1M, 3M and 4M on Rt. 15 in Manchester.

Figures 5 and 6 show the condition of beam-type dowels after the concrete blocks had been subjected to pull-apart tests and the dowels removed from the block. In all other cases also, the dowels showed severe rusting particularly in the area of the joint opening. The wet and soggy condition of the fiber-type joint filler, which acts like a wick and provides a constant source of moisture around the dowel, is probably the greatest contributing factor to the severe rusting of the dowels at the joint opening.

Figure 7 shows the condition of the formed intermediate joints 2M and 5M. Note the complete failure of the joint seal in this joint as compared to the joint seal in the expansion joint of Figure 3. Reference to Figure 1 indicates that intermediate joints 2M and 5M are 147 ft apart with two expansion joints 3M and 4M between them. Figure 8 shows the extent of the opening of intermediate joint 2M which is also representative of intermediate joint 5M. The reinforcing steel through each of these joints was found broken and, judging by the rusted condition of the fractured ends, the steel had been broken for some time. In the 1,000 ft of concrete pavement immediately preceding the area where the sample blocks had been cut out, 7 of 30 formed intermediate joints have opened abnormally in either one or both lanes.

In Figure 9 the ends of the broken reinforcing steel may, perhaps, be seen with the aid of a magnifying glass. The extent to which the sealing material has flowed down into the joint may also be observed.

Figure 2, previously discussed, shows the well-sealed condition of expansion joint 1U on Route 15, Union, at which a test block was removed. At expansion joint 5U there was evidence of considerable bond or adhesive failure. In the pull-apart tests, a load

Figure 4. Condition of joint filler in expansion joints 1M, 3M and 4M. The original filler was 1/2 in. thick. In the above joints it measures 1/8 in. in joints 1M and 4M and 1/4 in. in joint 3M.

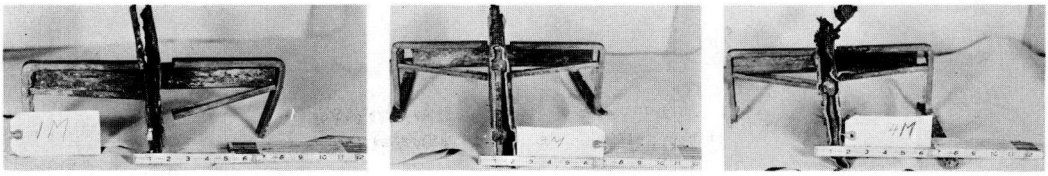

Figure 5. Condition of load transfer dowels in expansion joints 1M, 3M and 4M.

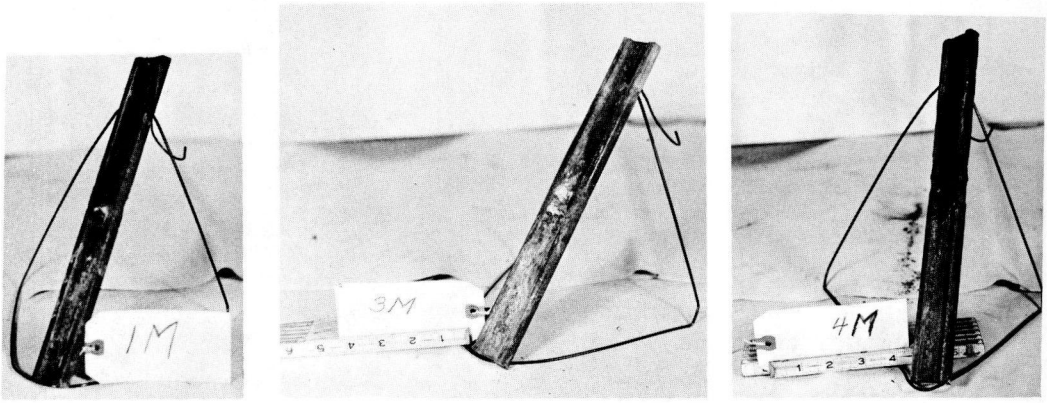

Figure 6. Rust and erosion of dowels from expansion joints 1M, 3M and 4M.

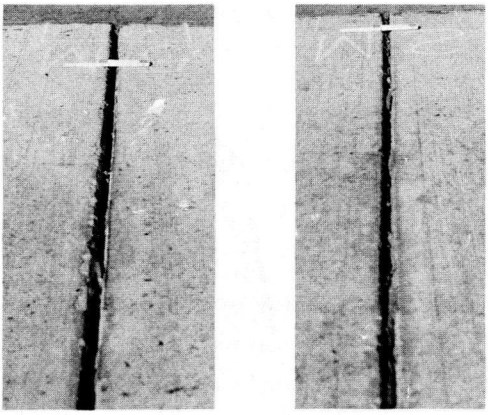

Figure 7. Condition of joint seal and opening of intermediate formed joints 2M and 5M on Rt. 15 in Manchester.

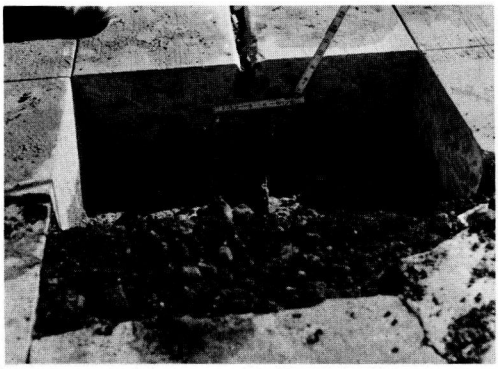

Figure 8. Width of opening at intermediate joint 2M. The longitudinal steel of the reinforcing mat is broken and joint has opened 5/8 in.

of 7,900 lb did not cause any movement at expansion joint 1U, but at joint 5U a load of 100 lb pulled this joint completely apart. It is quite evident, therefore, that in judging the merits of a joint seal in any expansion joint on the basis of appearance it is important to ascertain whether or not the joint is functioning properly.

The original ¾-in. non-extruding-type joint filler installed in this pavement in 1953 has, after 3 years of service, been compressed to ½ in. as indicated in Figure 10.

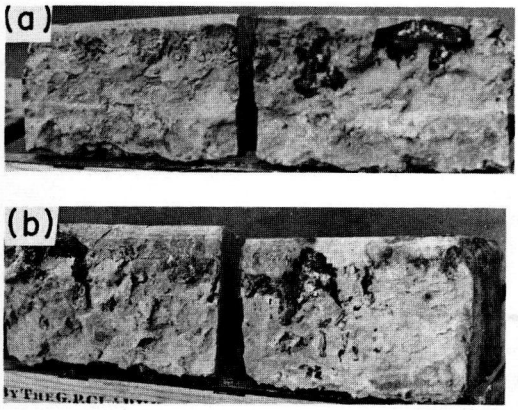

Figure 9. Joint faces and the ends of the broken longitudinal steel—(a) intermediate formed joint 2M and (b) intermediate formed joint 5M.

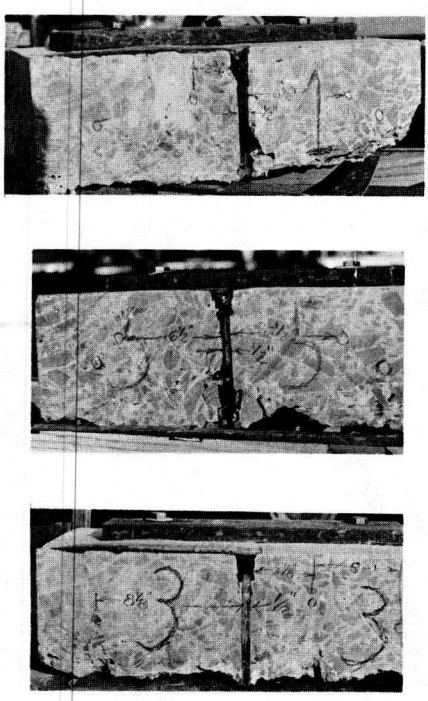

Figure 10. Expansion joint filler originally 3/4 in. non-extruding fiber filler. It now measures 1/2 in.

The condition of round dowel 1U (Fig. 11) after removal from the test block indicates considerable rusting at the joint opening decreasing in severity along the sliding end of the dowel. These dowels were plain round steel dowels with no coatings, except for greasing of the sliding end. Except where concrete was noted to be porous, the fixed end of the dowel shows no indication of rust progressing along this end. Here again the water-soaked joint filler is probably responsible to a large

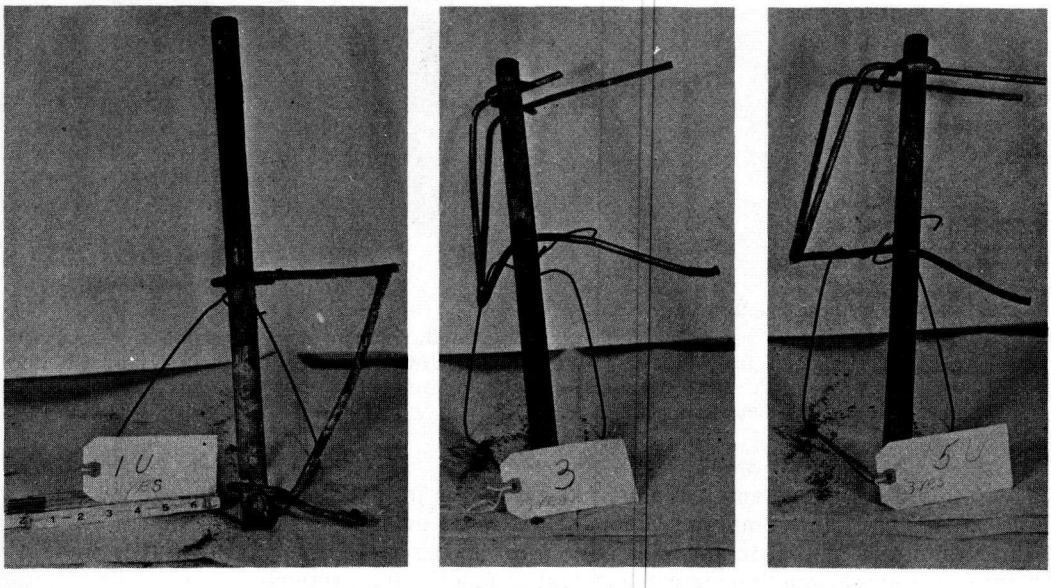

Figure 11. View of dowels after removal from concrete. Some rust has formed at the opening. The dowels were wire brushed to determine loss of section due to rust.

extent for the formation of rust on the sliding portion of this type of dowel.

Mention was previously made of the fact that it required a load of only 100 lb to open the expansion joint of test block 5U. Figure 12 shows why this small load opened this joint completely. The longitudinal section of the concrete pavement encasing the sliding end of the dowel is a good example of poor consolidation of the concrete around the dowel and lack of intimate contact essential to an efficient transfer of load across the joint. This fact may account for the occasional faulting noted in the expansion joints throughout this project. From the appearance of the joint faces of specimens 1U and 5U (Fig. 12) the wire assembly holding the dowel tends to prevent thorough encasement of the entire dowel assembly by the concrete.

Figure 13 shows the wide open condition of the sawed intermediate joints 2U and 4U. The respective locations of sawed intermediate joints and the expansion joints involved are shown in Figure 1 on the eastbound roadway of Route 15, Union. Joint 4U in Figure 13 is interesting because it definitely shows that in two lanes of concrete, with longitudinal tie bars on 5-ft centers, there can be limited movement in one lane without affecting the adjacent lane. The sawed joint in the far lane appears to have undergone no change in width whereas the sawed joint in the near lane has opened to ¾ in. There is, however, some evidence of distress along the longitudinal joint adjacent to the transverse expansion joint due to the differential lane movement. On removal of these test blocks from the pavement, it was found that, at the time of construction, the longitudinal reinforcing steel had been sawed completely through at joint 2U and 75 percent through the steel in joint 4U. While this was true in the aforementioned two intermediate joints, it has been observed in two cases, where other sawed joints pulled apart on this project, that the steel appeared to be broken rather than sawed. In the latter two cases no test blocks were removed from the pavement but the open joint was blown out with compressed air, and the ends of the steel reinforcement were observed in the open joint with the aid of a flashlight.

Inspection of 1,300 ft of pavement, including the area in which test blocks were removed from both the expansion joints and the sawed intermediate joints, disclosed that 10 sawed joints of 39 had opened abnormally.

Basically, the concrete pavement design used throughout Route 15 provided for expansion joints at approximately 100-ft intervals and either formed or sawed intermediate joints at approximately 25-ft intervals with the reinforcing steel carried through the intermediate joints. These intermediate joints are in effect predetermined cracks in the concrete pavement and, as such, they should be held tightly together.

The concrete slabs in the eastbound roadway of a test road constructed in 1948 at Portland, Conn., varied in length between expansion joints from 96 ft to 172 ft. There were no intermediate joints in this pavement and the reinforcing steel, which was of the same weight and cross-section as that used on Route 15, was held constant throughout. In October 1950, one crack appeared in the 154-ft slab. To the end of 1956, no further cracking was noted in any of the other slabs. When first observed, this crack was very fine and appeared to end at a point

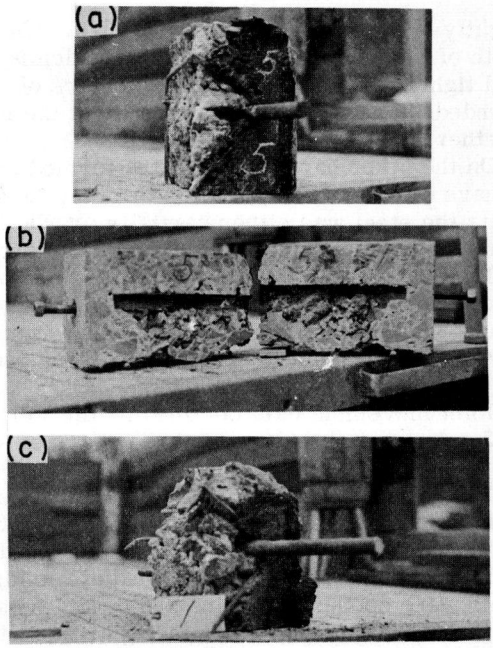

Figure 12. Upper photos (a) and (b) show poor consolidation of concrete around the dowel slip end from joint 5U. Lower photo (c) shows poor consolidation of concrete around the fixed dowel end at joint 1U.

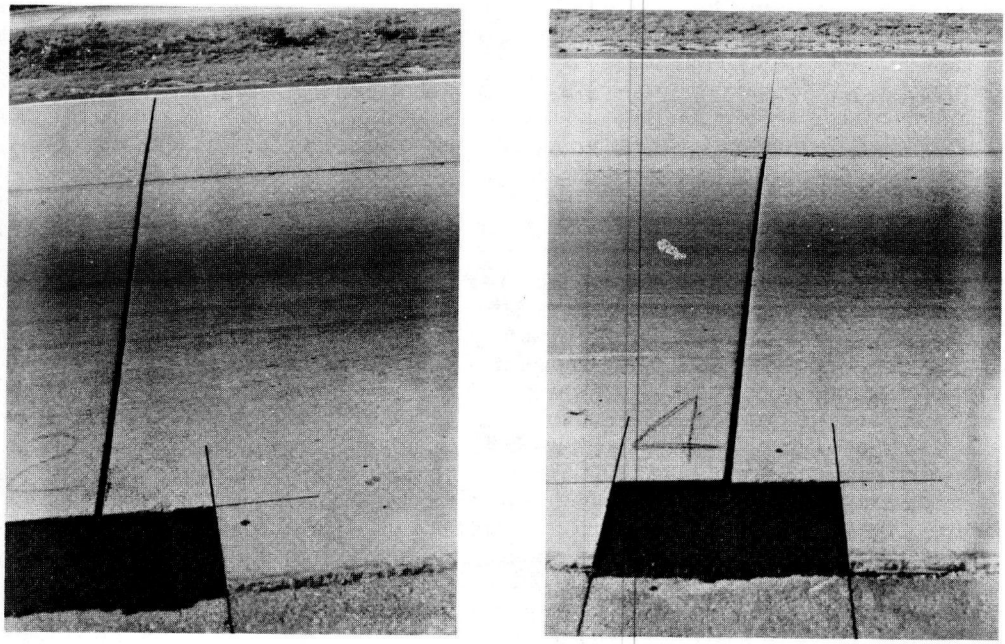

Figure 13. Intermediate sawed joints on Rt. 15 in Union. Note comparative widths of sawed joint #4 across the pavement. Joints were sawed originally full width of pavement.

slightly over half-way across the slab. This crack now is plainly visible for the full width of the slab and appears to be widening slightly, although the crack faces are still held tightly together. After eight years of service this pavement is still functioning as intended. One crack has appeared but the reinforcing steel is holding the faces tightly together.

On the other hand, many of the formed and the sawed intermediate joints on Route 15 have cracked and opened up to ¼ in. to ¾ in. Undoubtedly in a number of these joints the steel was either partially or wholly sawed through, in the case of the sawed joints, which accounts for the numerous excessive intermediate joint openings at the early age of 3 years. Where the intermediate joints were formed, however, there is no possibility of the steel being prematurely weakened by sawing. Although no information is available as to when the formed joints opened and the reinforcing steel broke, it seems reasonable to assume that this condition occurred at a much later date than in the case of the sawed joints.

Joint movement data obtained on the Vernon test section on Route 15, constructed in 1952, showed, at the end of three years of service, that 15 sawed intermediate joints had opened an average of 0.015 in. Two of the 15 joints observed had each opened 0.030 and 0.022 in., respectively. At the end of 4 years of service the average opening of the 15 sawed intermediate joints was 0.033 in., the aforementioned two joints had each opened to 0.12 in. and a third joint had opened to 0.04 in. In January 1959, the average opening of the 15 intermediate joints was 0.12 in. The two aforementioned joints were opened 0.604 in. and 0.426 in. and the third joint was opened 0.463 in. Very obvious faulting is occurring at these three joints. Movement at the expansion joints nearest to these intermediate joints has become very small due either to dowel restraint, or to shortening of slab lengths due to breaking of steel reinforcement, or a combination of both. On the basis of this data it appears that some of the sawed joints may be expected to open appreciably after three years of service.

Based on tests conducted by the Bureau of Public Roads, Sutherland and Cashell have stated that effective stress control due to aggregate interlock at a joint is not dependable

when the joints are open 0.04 in., or more, irrespective of the type of aggregate used. Therefore, at the intermediate joints with openings of 0.10 in., or more, there are in effect free pavement ends. Due to the deflection of these free ends under heavy loads, the flexural stresses plus the tensile stress due to reduction of pavement temperature may become sufficient to break the steel completely.

The wire mesh reinforcement removed from the test blocks cut out in both Manchester and Union on Route 15 was checked by the laboratory for compliance with the standards and specifications applying at the time of construction of this pavement. The steel was found to comply in all respects with the requirements at that time. The range in breaking strength of the steel tested varied from 75,050 psi to 84,280 psi. The minimum tensile strength allowable per ASTM A82 is 70,000 psi with the yield point as 0.8 the tensile strength. Assuming the average tensile strength of the wire mesh as used on Route 15 was 80,000 psi, the yield strength would be 0.8 of 80,000 psi or 64,000 psi. Since the area of steel reinforcement for a 12-ft lane is 0.135 percent of the concrete area, the load required to develop the unit stress of 64,000 psi is equal to 8,320 lb per foot of width of concrete pavement. In a concrete slab 100 ft in length, and with no restraint to movement other than subgrade friction, the pull developed at the midpoint of the slab during a period of contraction of the concrete might very well be 5,000 lb per foot of width of concrete, assuming a coefficient of friction of 1.0. These figures are considered quite conservative. For the foregoing conditions, the stress developed in the steel, assuming the concrete had cracked, would be well below the actual yield point of the steel and any cracks which might occur should, under these conditions, remain tightly closed. As will be shown, however, the ends of the concrete slab are restrained from moving due either to rusting of the dowels, misalignment or a combination of both. With dowel restraint of such magnitude as to require a pull of 3,000 lb per foot of pavement to open the joint 0.1 in. (which is equal to 20 F change in pavement temperature) the total load placed on the steel, assuming the concrete has cracked, is now 8,000 lb. This leaves a very small margin to reach the yield point of 8,320 lb, which might very well be exceeded when a heavy truck passes over the crack and causes a slight flexing of the steel.

Laboratory

To determine the extent of the dowel restraint, the concrete blocks each containing one dowel were set up in a laboratory testing machine. A load was applied to the ends of the concrete through a steel yoke attached to the concrete with 5/8-in. steel bolts. Lead wool was compacted around the bolts in the concrete. This method of holding the bolts in the concrete was not very satisfactory as the bolts tended to pull out under loads over 5,000 lb. Two Ames dial gages were attached to the concrete at the joint in order to read the amount of opening as the load was applied. Unfortunately, the joint openings under load were too erratic to obtain the load required for any particular measured amount of opening, because in some cases, considerable load might be applied and no appreciable movement occur until the dowel restraint was overcome and then considerable movement might occur. The results of the pull-apart tests on the blocks removed in Manchester, and which contained the beam-type dowel, were as follows:

1. A load of 1,500 lb was required to open joint 1M 0.1 in. and a load of 10,000 lb to open the joint 0.34 in. An opening of 0.1 in. is equivalent to a change of 20 F in pavement temperature and an opening of 0.34 in. is equal to a change of 55 F in pavement temperature. Maximum recorded seasonal changes in pavement temperatures vary from 80 F to 100 F.
2. A load of 7,000 lb was required to open expansion joint 3M 0.1 in. and further loading had to be suspended due to cracking of concrete around the yoke bolts.
3. Joint 4M opened suddenly after a load of 5,700 lb had been applied. The load was increased up to 8,300 lb with a joint opening of 0.52 in.

The following are the results of the pull-apart tests on the expansion joints removed in Union and which contained the round-type dowel.

1. Joint 1U would not open under a load of 7,900 lb. Further loading was not applied because the yoke bolts began to pull out.

2. A load of 3,700 lb opened joint 3U 0.1 in. and at 3,900 lb the dowel pulled out of the concrete.

3. Joint 5U practically fell apart under a load of 100 lb. As previously mentioned this was due to poor consolidation of concrete around the dowel.

Discussion

Although it is improbable that the test results shown for single load transfer dowels apply equally to all the dowels in the complete load transfer assembly of a lane, it is desirable to project these test results for the full joint length, even though somewhat questionable assumptions are made.

In a 12-ft lane of 8-in. concrete pavement there are installed 10 beam-type dowels or 12 of the round-type dowels. On the basis of the data obtained in the pull-apart tests, restraint to movement offered by either type of dowel may very well require an average pull of 5,000 lb per dowel to open a joint 0.1 in. For the beam-type dowels the total pull in a 12-ft lane would be 50,000 lb or for the round-type dowel 60,000 lb. This is equal to 4,167 lb and 5,000 lb, respectively, per foot of width of concrete. Assuming that the point of zero movement due to temperature change is at the center of the slab, the weight of a longitudinal 1-ft strip of concrete with an expansion joint spacing of 100 ft, which must be overcome during a period of contraction of the concrete, is equal to 5,000 lb. The total force required to move the concrete 0.1 in. would then be 5,000 lb plus 4,167 lb or 9,167 lb per foot of width of concrete for the beam-type dowel, and 10,000 lb per foot of width of concrete for the round dowel. As previously stated, this force combined with tensile stress induced by heavy loads during the winter months is sufficient to break the longitudinal reinforcing steel. Where the steel was partially cut in the process of sawing the intermediate joints, a considerably lower force could break the reinforcement at an earlier age.

In summary the following facts are cited:

1. The intermediate joints in those pavements which provide an expansion joint spacing of approximately 99 ft to 100 ft are cracking and are gradually opening from ¼ in. to ¾ in. despite the use of reinforcement through the joint which should maintain a tight crack.

2. In general this condition occurs at the central intermediate joint, although there are instances where the joint adjacent to the expansion joint has opened considerably rather than the central joint.

3. The reinforcement through the formed intermediate joints is broken, and plainly visible joint faulting is occurring. The reinforcement through the sawed intermediate joints had been either partially or totally cut during the process of sawing the joints. Based on data obtained of the movement at the sawed joints on the Vernon test section, an opening of ⅛ in. was observed at two of the sawed joints at the end of three years. Variations in the degree of joint opening are probably related to the extent to which the steel was cut during the sawing of joints, and dowel restraint to movement.

4. There is considerable restraint to movement at the joint, due, it is believed, to rust on the dowels at the joint opening which tends to progress along the sliding end of the dowels. Based on the data obtained in the pull-apart tests of small joint sections cut out of the pavement, this restraint varies from 3,000 lb to over 8,000 lb per dowel. The joint filler retains the moisture to a very large degree and maintains a continuously wet condition around the dowel across the joint.

5. Notwithstanding the evidence herein, the long slabs without intermediate joints constructed on the Portland test road in 1948 have, after 8 years of service, developed but one single crack which is, however, held tightly together. The same unit area of steel reinforcement per foot of width of concrete was installed in this pavement as was used in the concrete pavement of Route 15.

In view of these facts it is concluded that the degree of restrained movement due to dowel corrosion is sufficient to cause the reinforcing steel to break and the intermediate joints to open much wider than was anticipated. Due to failure of the seal in these joints, sand and other incompressible materials have infiltrated into these joints and

caused them to remain open. Consequently, during period of hot weather the expansion joints have been subjected to considerable compressive force which has gradually reduced the thickness and effectiveness of the joint filler. While the partially sawed steel is responsible for more frequent and wider opening of the intermediate joints at an early age, it is quite apparent that this condition will eventually prevail even where these joints are formed. This condition appears mainly in the pavement design which provides for an expansion joint spacing of 99 to 100 ft with equally spaced intermediate joints.

EXPERIMENTAL WORK

Early reports on the foregoing observations and tests led to the decision to explore the possibilities of rust resistant dowels. For round dowels, sleeves of rust resistant metals constituted a possible solution. For the beam-type dowel, some other means of applying a protective coating seemed indicated, and some of the more modern materials in this line held promise.

In early 1957, the cooperation of a number of manufacturers of dowels and coating materials was obtained. Time for testing was restricted by the decision to make experimental dowel installations on the Connecticut Turnpike, which was then in part in the grading stage.

Laboratory

Arrangements were made with a commercial laboratory to proceed with a salt spray test on specimens submitted, to be followed by Taber abrasion test on standard specimen plates coated with the more promising protective coatings. While there might be a question as to the choice of tests, they had the advantage of being standard tests which could be completed during the available time.

Submitted for the salt spray test were 21 specimens composed of 19 round dowels and beam dowels protected with 12 different coatings, one round dowel partly sleeved with stainless steel, one 431 heat treated stainless steel dowel beam and one each Parkerized and plain untreated dowel beams.

The conditions of test were reported as follows:

PH	6.9
Temperature	95 F
Specific gravity of solution	1.140
Fog collected (ml/hr)	0.65

Observations were made at 24-hr intervals and the test was stopped after 2,088 hr. Some of the specimens were withdrawn before the completion of the test because advanced rust formation was evident. Early rusting was observed on all unprotected steel surfaces and on the areas exposed by an X scratched through the coatings.

The Taber abrasion test was performed on plates representing 11 of the coatings which showed apparent promise, of these only five withstood 4,000 cycles.

There was concern about the ability of these coatings to withstand rough handling during construction. A crude, manual impact test eliminated another coating which demonstrated extremely brittle characteristics.

Field Installations

On the basis of these tests the four coatings, which are patented commercial processes, showing best performance were selected for application on beam dowel assemblies to be installed on the Connecticut Turnpike in Greenwich. As it were, a 40-ft contraction joint design had been selected for this pavement in preference to the 100-ft slab with intermediate joints.

Sufficient beam dowels were coated to provide for four groups of 25 contraction joint assemblies for the three 12-ft lanes of 9- and 10- in. pavement. Bond breaking grease was applied to the sliding end of all dowels prior to shipment to the job site. Installation of the experimental dowels was completed by November 1957. Plugs for measurement

of joint movement were installed at the shoulder edge of the pavement at all joints containing experimental beam dowels and in a control section of 25 joints with untreated beam dowels.

In May 1957, a considerable number of contraction joint load transfer assemblies with round dowels partly sleeved with stainless steel were installed on the Connecticut Turnpike, Milford. Here also plugs were placed at 25 joints each of the sleeved and regular dowels.

At a third location on the Connecticut Turnpike, Old Lyme, load transfer assemblies containing round dowels, partly sleeved with Monel metal, were installed in twelve 24-ft contraction joints in May 1958.

A fourth installation comprises 12 contraction joints, 24 ft long, which contain load transfer assemblies with round steel dowels, nickel coated or clad for their full length. This installation was completed on Route 9, Middletown, in August 1958. On these last two installations plugs were also placed at the experimental joints and at control joints.

On the basis of measurement of movement occurring at the joints containing experimental as well as control load transfer assemblies, there is, to date, no marked difference or trend in the performance as far as unrestrained movement is concerned.

Accelerated Test

When the load transfer assemblies containing the variously coated beam dowels were manufactured, it was requested that additional coated beam dowels be furnished for further tests. So far the only other test attempted consists of exposure of such dowels to tidal waters. For this purpose, three each of the beam dowels protected with the four different coatings, two monel sleeved round dowels, and three nickel clad round dowels were partly encased in concrete in "dumb-bell" fashion. A piece of reinforcing rod was cast in, parallel to the dowels, so that the assemblies could be lined up on a supporting rod to prevent casual removal. These dowels were placed in the tidal range of the Black River at Old Saybrook on April 16, 1959.

The dowels were examined on September 24, 1959 after five months of exposure to the salt water. The nickel coated dowels showed no signs of rust. The two monel clad dowels also showed no signs of rust although there is evidence of slight pitting of the metal sleeve. The remaining dowels, which are of the beam type and were coated with various corrosion resistant materials, show very slight signs of rust. One of three dowels coated with the same material did show about 1 sq in. of rust on the web whereas the remaining two showed only tiny spots.

In general the steel channels in which the beam-type dowels slide and which were also coated with corrosion resistant material, show slight signs of rusting at the outside corner edges of the channels. It appears that the coating may not be sufficiently thick at these 90-deg corners.

At the present time the nickel coated dowel appears to hold considerable promise of a rust free dowel. Any conclusive evidence of a superior rust resisting quality between the various coatings used has not been noticed at this time.

Radiography

There has been some speculation as to what extent, if any, radiography might help in the detection of dowel corrosion. In turn, there is a question whether or not radiography can distinguish between steel and iron oxide unless there were a considerable loss of section. The Department was fortunate in obtaining the cooperation of a manufacturer of radiographic equipment in trying to find an answer to this last question. Radiographs of one each of the encased nickel clad and the partly monel sleeved round steel dowels were taken before they were placed in the tidal water. Periodic radiographs will have to be taken hereafter.

CONCLUDING COMMENTS

While the effect of dowel corrosion as, at least, a contributory cause of malfunction of expansion joints had been established satisfactorily, the magnitude of this effect in contraction joints is still somewhat in doubt. It is hoped that the accelerated test will

allow evaluation of the various protective applications reasonably soon. As to their performance in the pavement, an answer may not be as near at hand.

The selection of the Connecticut Turnpike for the experimental installations was a good one, as far as heavy traffic and load applications are concerned. It proved to be a bad one, however, as concerns any attempts to remove dowel blocks from the pavement. Even the taking of measurements at joints is disturbing to the State Police.

It is to be hoped that research by others will help to create a better understanding of the dowel corrosion problem, and that definite standards for corrosion resistant applications, of whatever nature, may ultimately derive therefrom.

ACKNOWLEDGMENTS

The author wishes to acknowledge the assistance given in the preparation of this paper by F. Sternberg and O.A. Strassenmeyer of the Division of Research and Development. Gratitude is also expressed to the various cooperating manufacturing concerns.