JOINTED CONCRETE PAVEMENTS IN MICHIGAN
DESIGN PERFORMANCE AND REPAIR

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The report covers background information concerning the performance and problems related to postwar pavements with 99-ft reinforced slabs, load transfer, and base plates under the joints. Newer pavements have been designed with successively shorter slab lengths. Load transfer and reinforcement still are used. A major installation of experimental pavement is discussed. Experience to date has resulted in a recent change to incorporate free draining base materials with subbase drains in future Interstate projects. The first concrete recycling job has been indefinitely delayed because of fund reductions. The effects of pressure build-up in older pavements is discussed, along with strategies for pressure relief, experimental pressure relief projects, preventive maintenance, and the development of techniques for locating and installing joint filler in pressure relief joints. The effects of base drainage on the performance of the concrete pavement as well as the inter-relationships with aggregate quality, are demonstrated for an experimental installation having extreme variations in drainability. A few details of concrete shoulder design and some examples of compatible slab length considerations are suggested, along with brief comments on corrosion resistant load transfer dowels.

Background

Postwar portland cement concrete jointed pavements on State trunklines in Michigan were reinforced with 30.2-m (99 ft) slab length and dowel bars for load transfer. Most were 23 cm (9 in.) thick. Joints were formed and relatively narrow with poured sealants. Base plates were installed beneath the load transfer assembly to support it during construction operations, to prevent infiltration of base materials into the bottom of the joints, and were turned up at the ends to keep shoulder materials out of the joints. This design was used during the years that much of the Interstate system was constructed, and pavements of this type now are in place on roughly 80 percent of the major concrete highways in the state.

It soon became evident that the ineffective joint seals allowed moisture, salt, and incompressible materials to collect in the joints above the base plates. These factors--along with weather and the associated factors related to the concrete aggregates--caused crumbling of the lower portion of the joint faces, leaving little area to resist compressive forces that built up in the pavements. The result was a serious problem with joint failures and 'blow-ups.'

During the 1960's, sawing of joint grooves was introduced, along with preformed neoprene seals; slab lengths were reduced to approximately 21.3 m (70 ft) and base plates were eliminated.

In 1971, the Research Laboratory recommended a preventive maintenance concept for the old (30.2 m slab) pavements (1). During the 1970's, slab lengths on new pavements were reduced still further, to approximately 12 m (40 ft), designs of the neoprene seals and joint grooves were improved, and load transfer dowels were required to have protective coatings. Moreover, slipform pavers and central mix concrete plants came to be used almost exclusively for construction.

An extensive experimental pavement installation was built in 1975 on US 10 relocation north of Clare (2, 3). This project included sections using an unusually large variation of drainability in the base, ranging from zero to several hundred meters per day. Performance of these experimental pavements over the first five years is covered later in this report.

Early in 1980, standards and specifications for base materials on Interstate routes were revised to provide considerable increases in drainability. Present requirements are for a 10-cm (4 in.) layer of base material to be open-graded. Either 100 percent crushed, unstabilized material, or more rounded material with asphalt stabilization, may be used at the contractor's option.

Improved centerline seals, sealed longitudinal bulkhead joints, and larger (16 mm or No. 5 rebar) lane tie bars with epoxy coating have been added. Therefore, the main item requiring further attention for concrete pavements is the quality of aggregates. This item is under discussion at the present time.

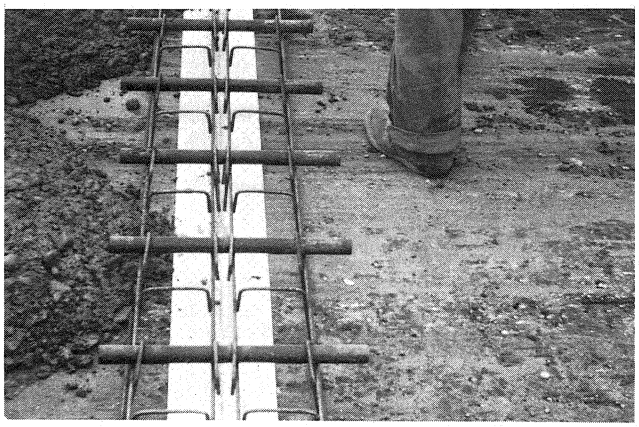
In 1980, plans and specifications were drawn for the first recycling of portland cement concrete

pavements on a state route. Letting has been delayed indefinitely due to reductions in funding. This project was to be located on US 23 just south of I 75 in Genesee County.

Compressive Stress Build-Up in Older Pavements

Base plates under the joints in the older pavements accelerated the deterioration of the lower faces of the joints as shown in Figure 1.

Figure 1. Base plates in new construction (top) were placed to keep aggregates out of the joint. The photo at bottom, however, shows the long-term effects of increased joint face deterioration that resulted when salt and moisture were trapped in the joints.

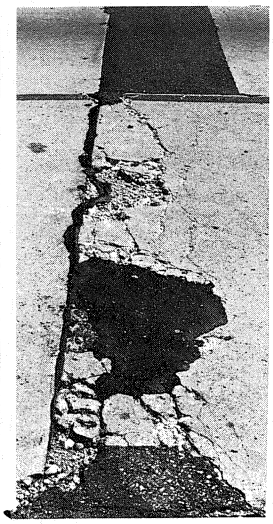




Ineffective joint seals allowed infiltration of incompressible materials and deicing chemicals into the joints when the slabs contracted during cold weather. Subsequent expansion of the slabs caused by spring's moisture and high temperatures caused crushing of the joint faces and, in many cases, severe joint blow-ups occurred (Fig. 2). These caused traffic disruptions and required emergency repairs that were quite expensive. This type of failure in the older pavements usually occurred at about 12 to 16 years after construction. The same compressive forces sometimes caused damage to bridges. Pressure relief joints about 10 cm (4 in.) wide, filled with foamed-in-place urethane, were installed in bridge approach slabs in the Detroit area during the late 1960's. Subsequent discussions within the Department involved the use of such joints at intervals inpavements to prevent blow-ups.

Figure 2. Joint crushing (left) or sometimes severe blow-ups (right) developed at the deteriorated joints when hot spring and summer weather caused expansion of the pavement slabs.



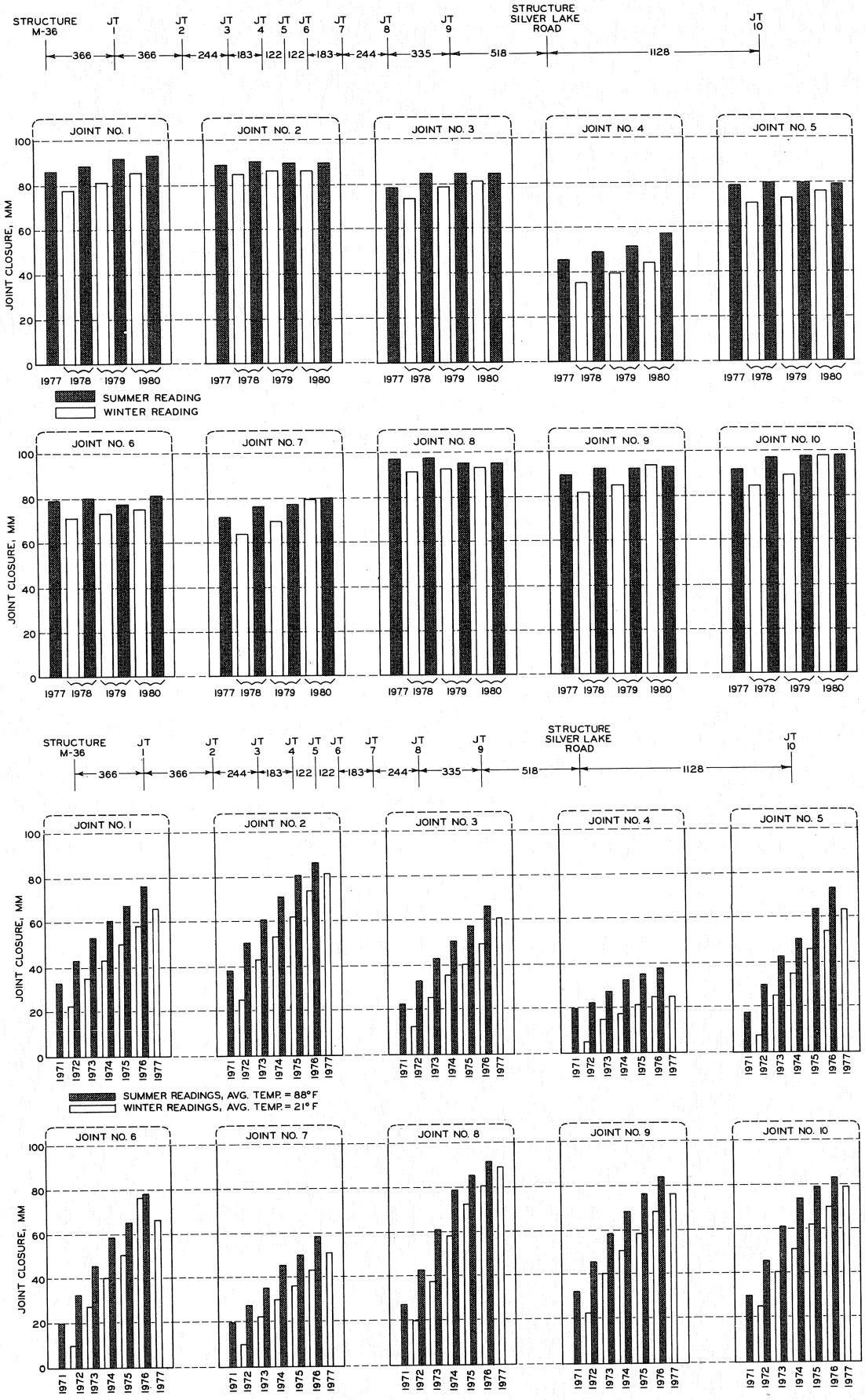


The length of pavement that can be relieved for a given length of time, by a pressure relief joint, is entirely dependent upon the condition of the adjacent joints and the amount of stress in the pavement. Forces generated in pavements can be extremely high and a very weak joint could blow even if a relief joint were nearby. Consider that it takes approximately 250,000 lb of force to move one 30.2-m (99 ft) by 7.3-m (24 ft) slab, and each additional slab adds another such increment of load. It can be seen, therefore, that very high forces, indeed, would be required to move several hundred feet of pavement towards the relief point. If pavement joints adjacent to a newly installed compressible joint are badly deteriorated, it is possible for additional blow-ups to occur within a few hundred feet of the new joint. Therefore, the worse the joint condition, the more closely spaced the pressure relief joints would have to be to eliminate blow-ups for a given length of time.

During the spring of 1971, the Department's Research Laboratory and Maintenance Divison cooperated in an experimental installation of pressure relief joints, to determine their effectiveness in reducing joint blow-ups in a 14-year old pavement that was showing signs of joint deterioration and high compressive stress. Nine joints were installed in about 2,680 m (8,800 ft) of pavement, with spacing varied from 122 m (400 ft) to 305 m (1,000 ft). An adjacent roadway at the same site, in similar condition for joint width measurements was installed and pavement ratings were made. Repeated surveys made over the lift of the project have shown no serious blow-ups in the relieved section of roadway, while numerous repairs of deteriorated joints have been made in the adjacent control section.

Results of the pressure relief joint width measurements are shown in Figure 3. The data show quite rapid closure of the joints, so that much of the expansion space provided is utilized by the pavement during the first few years after installation. A corresponding increase in width occurs at nearby joints and 'working' cracks. Note that this pavement was one without effective seals, and with several full-width cracks where the reinforcement had failed; such joints should not be placed in

Figure 3. Location of pressure relief joints and total closure - US 23.



pavements with preformed compression type seals, as joint opening will allow the adjacent seals to come out.

Joint filler material used on this and other early pressure relief projects was ethafoam. The filler was placed in the spring so that expanding pavement would compress it in place. Problems were encountered later when contractor-installed joints had fillers float out during cool, wet weather. Laboratory investigations showed some physical changes in later samples of foam from that previously used, and also indicated marginal recovery after compression for both types of ethafoam. The same experiments showed good recovery after compression for urethane foam, even when saturated and frozen. Therefore, specifications were changed to require the use of urethane fillers for all subsequent pressure relief joint work. A liquid urethane adhesive applied to the concrete joint faces also is used, and results with the newer filler have been very good.

It became obvious during the early work that relief joint fillers should be placed under some precompression so that contraction of the pavement slabs would not leave the filler loose in the joint. Therefore, the Research Laboratory developed a machine which attached to the front of a snowplow truck, to place the 10-cm (4 in.) filler in 8-cm (3-1/4 in.) joints. Plans have been furnished to contractors and several machines of this type have been built for use throughout the state (4).

The Department was experiencing numerous problems with joint blow-ups during the late 1960's and early 1970's (as many as 1,000 or more per year reported) causing associated traffic disruptions and expensive emergency repairs. Based on the results of the experimental pressure relief project, along with a considerable amount of other work on precast repair slabs and development of the fast-setting cast-in-place concrete repair procedures (5, 6, 7), recommendations were made to begin a program of contract repairs on several major projects. The projects were selected where evidence of high compressive stress and some joint deterioration were present, but where few failures had yet occurred. The intent of the program was to prevent the occurrence of disruptive joint failures or the need for further major repair efforts, for a period of at least five years. Continuing evaluations of the projects have shown that the intended five-year interval has been attained for all practical purposes. State or county personnel have replaced a few joints and routine patrol patching is done by maintenance forces, but major disruptions of traffic and extensive additional repairs have been avoided. Some projects are in need of additional attention after five years' service while others seem to be in adequate condition for a few more years of service. The rates of increase in additional deterioration have been shown to vary considerably, and appear to depend upon construction contractor, source of materials, and truck traffic.

Please note that in all cases here, we are discussing repairs of the older 30.2-m (99 ft) slab length pavements with poured sealants and base plates. Experimental work on these projects resulted in the refinement of an existing rating system so that joint conditions could be quantified, and warrants for joint replacement or pressure relief could be established.

Various joint details were developed for use with repairs, so that contraction, 2.5 cm (1-in.), and 5 cm (2-in.) expansion details, were available. All joints were required to be sealed due to past experience with infiltration of incompressible materials and the corresponding pavement 'growth' or increasing compressive stress. Expansion space was placed either at joint repairs, or at mid-slab pressure relief joints. Limits were established so that at least 10 cm (4 in.) but not more than 15 cm (6 in.) of expansion space were provided every 259 m (850 ft). Joints are rated by estimating the amount of spalling exceeding 10 cm (4 in.) width, plus the amount of corner breaks, in linear meters along the joint. Warrants for replacement then were established. Usually 2 m (6 ft) or more of spall and corner breaks in a lane would warrant a lane joint replacement. (This value can be raised if limited funds present a problem or lowered if a more complete job is desired.) All repairs not full width of the pavement were made without expansion space. A minimum of 2 m (6 ft) of pavement removal parallel to centerline is required.

Although methods for reestablishing load transfer have been developed, all contract repairs to date have been without load transfer, because of cost. Such repairs do tend to fault under wet base conditions and heavy truck traffic.

Full-depth sawing with slab lift-out is required on repairs. Fast setting mix allows curing in four to seven hours so that repairs can be opened to traffic before dark when required. Mechanizing repair procedures has allowed contractors to increase significantly the number of repairs completed in a day. Approximately 100 lane joint repair slabs or more have been placed in a day on a job where deterioration was extensive and repairs were quite close together. Overnight lane closure was permitted on that particular project.

Effects of Concrete Aggregates and Base Drainability

Concrete pavement performance in Michigan has been improved considerably by base plate elimination, shortened slabs, and better seals. However, as could be expected, the quality of the materials used in the mix and the drainage provided in the base have significant effects as well. Although the state has numerous glacial deposits and rock outcroppings suitable for pavement concrete, the materials readily available in some parts of the state are subject to progressive deterioration. Figures 4 through 9 show pavement joints and cores (shown upside down) that were removed from the various pavements in the fall of 1979 and the summer of 1980, ranging in age from 4 to 21-1/2 years. Note the core in Figure 4 was removed from an expansion joint while the others are from contraction joints. Under Michigan conditions, expansion joints tend to close during the first several years (perhaps seven years or so) of pavement life. Therefore, in the older long slab pavements with poured sealants, they tend to stay sealed better than contraction joints and do not deteriorate as much. Figure 5 shows cores from contraction joints nearby the expansion joint of Figure 4, for comparison. The contraction joint cores show the deterioration typical of some of the older long slab pavements with poured sealants and base plates when they reach 20 years of age. L. T. Oehler reported in 1971, the results of a limited survey (8) that showed the decreases in rate of deterioration that resulted from improved joint seals and elimination of base plates. A far more extensive evaluation by Bashore and Price (9) confirmed the above results, and also indicated the following: 1) improved lubricant adhesive should be used for neoprene seals; 2) that hot-poured rather than cold-poured sealants should be used for longitudinal joints; 3) that neoprene seals for transverse joints should be extended down the edges of the slab; 4) that two-stage sawing should be used for trans-

Figure 4. Typical joint and core from I 96 near Portland. Expansion joint, core (shown upside down) is typical of the condition of expansion joints in the area. Constructed in 1958, 15.2-m (50 ft) slab with base plates, as a relief section for continuously reinforced pavement. After 21 years' service on this rural Interstate route the concrete is in good condition full depth.

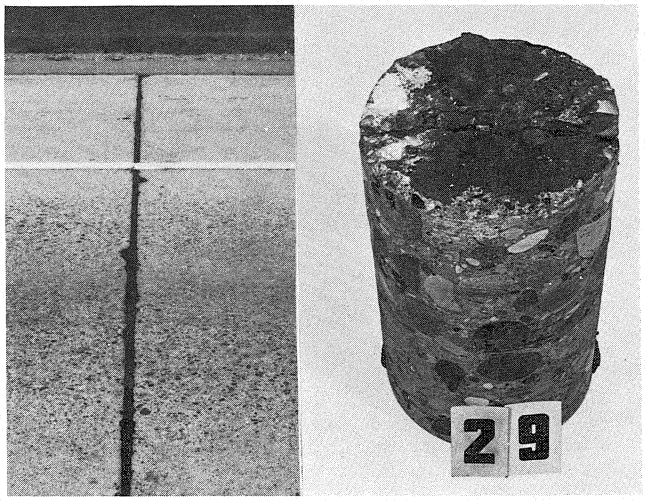


Figure 5. Joints and cores from I 96 near Portland. These are contraction joints (cores shown upside down), constructed 1958, 30.2-m (99 ft) slab with base plates. After 21-1/2 years' service on this rural Interstate route the joint faces are badly deteriorated and are fairly typical of pavements of this design and age.





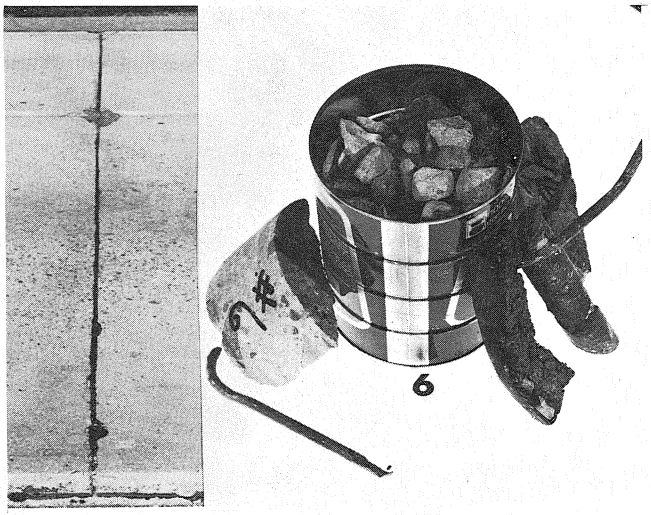


Figure 6. Typical joint and core from M 52 near Owosso. Contraction joint (core shown upside down) constructed 1969, 21.6-m (71 ft) slabs, no base plate, neoprene seal. After 10 years' service on this rural two-lane trunkline, minor deterioration is seen at bottom slab and horizontal cracks at bar level.

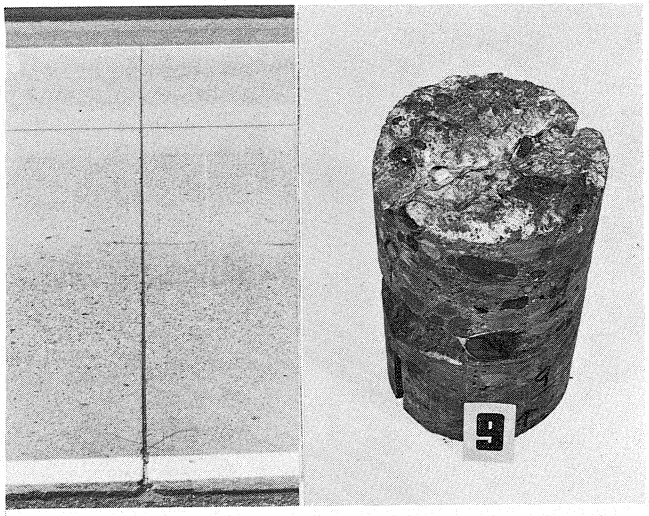
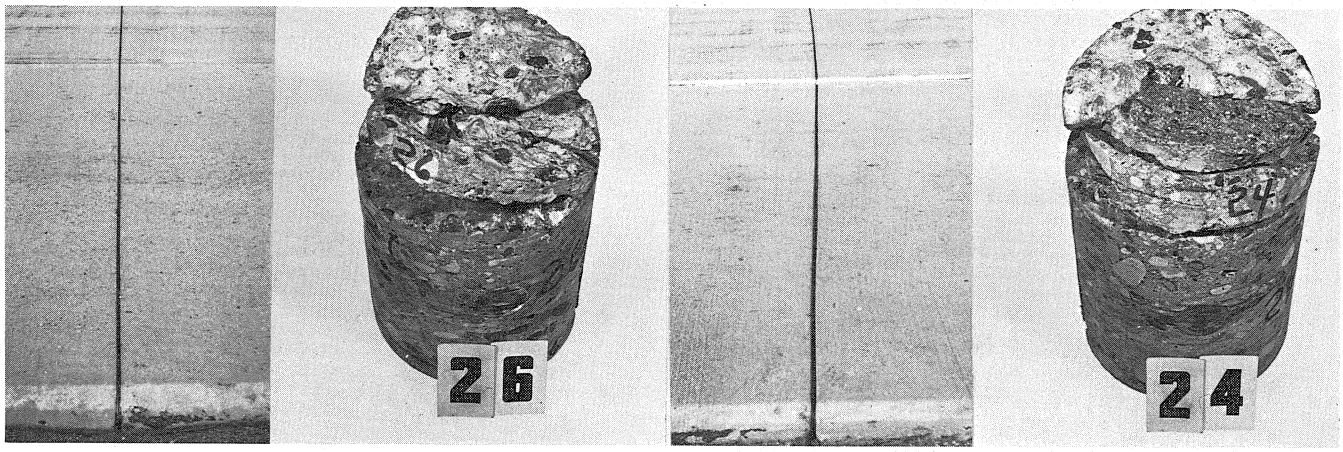


Figure 7. Contraction joints and cores (shown upside down) from I 75 near West Branch. Constructed 1973, specimen 26 is a 21.6-m (71 ft) slab, specimen 24 from a 17.4-m (57 ft) slab, and specimen 27 is from a 13.1-m (43 ft) slab, with no base plates and neoprene seals. This rural freeway had been in service for six years when sampled; all show moderately severe deterioration at the bottom of the joints.



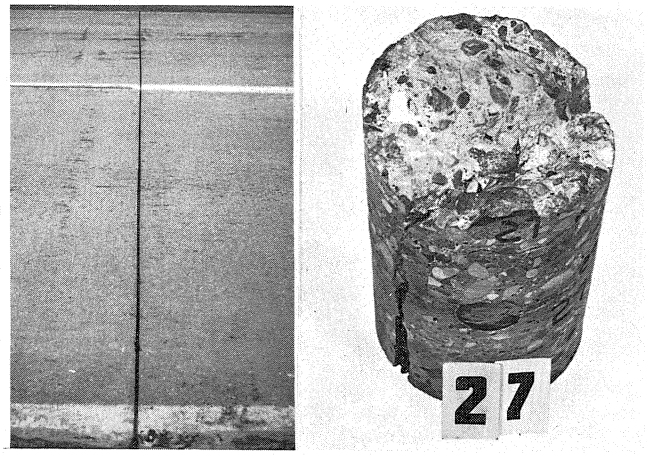


Figure 8. Contraction joint and core (shown upside down) from US 10 north of Clare. Constructed 1974 (but only in service four years when sampled), 21.6-m (71 ft) slab, no base plates, preformed neoprene seals. This rural freeway sample displayed moderate concrete deterioration at the bottom of the joint.

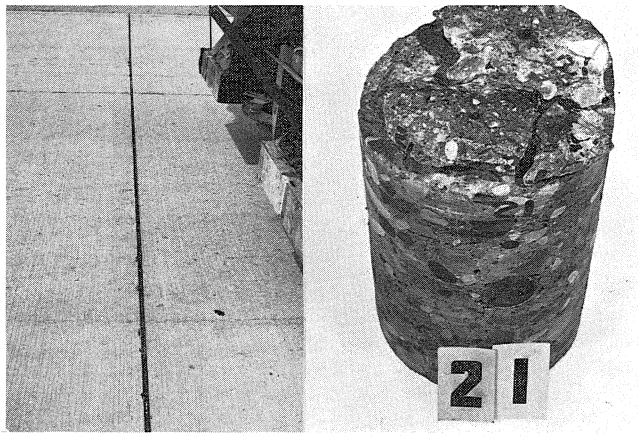
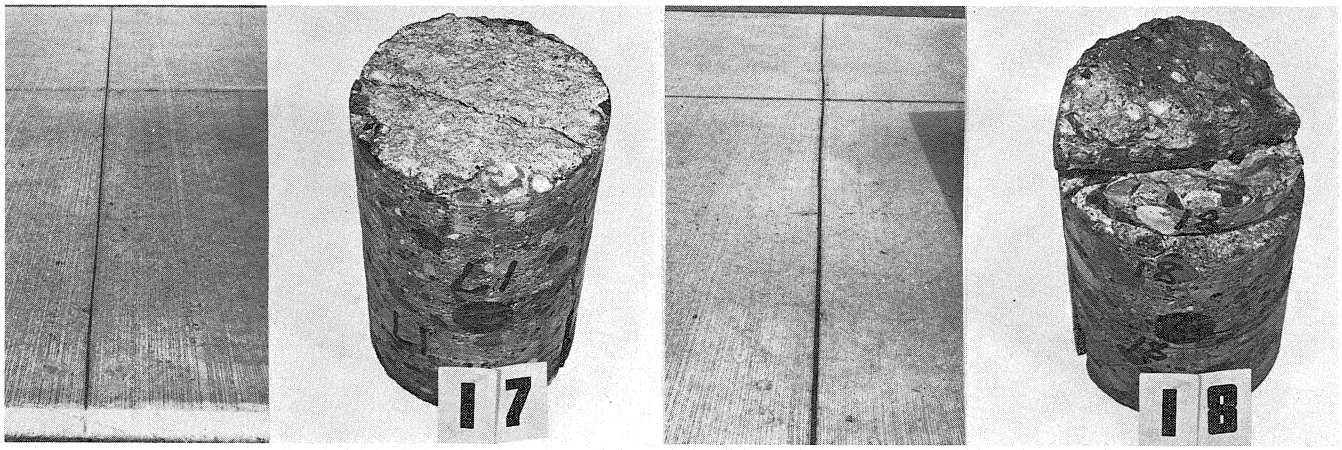


Figure 9. Contraction joints and cores (shown upside down) from the gravel base sections of the experimental installation on US 10 north of Clare. Constructed 1975 (in service four years), specimen 17 from 4 to 5.2-m (13 to 17 ft) slabs, specimen 18 from 21.6-m (71 ft) slabs of the control section, no base plates, with neoprene seals. Note the much greater concrete deterioration in the core from the longer slab from this rural freeway.



verse joint grooves; and, 5) that neoprene seals were effective in keeping incompressible materials out of the joints. However, all of the recent information shows clearly that the same general type of deterioration and cracking still are occurring at the bottom of the joints in the newer pavements, although at a considerably lower rate than in the older projects.

Figures 6 through 9 show deterioration of varying degrees at the bottoms of joints in some of the more modern projects. All of these pavements were placed on 10-cm (4 in.) densely graded base course over sand subbase, with preformed neoprene seals and without base plates. The type of deterioration is quite typical of that found throughout the state, and is quite similar for the four projects; although the projects are widely scattered geographically. Comparison of the core in Figure 9 from a short slab pavement on the same project as the longer slab pavement shown in the same figure, shows much better condition for the short slabs. The literature generally supports such better joint performance for short slab pavements. It is possible that the small sampling involved here is not adequate to determine the average condition for both types of pavement; however, further information will be developed as evaluation of the experimental project continues. Bashore and Price (9) found considerable variation in the amount of joint deterioration from one joint to another within a given project, as one might expect from local variations in construction, materials, drainage, and joint opening; therefore, the two undamaged cores may not be highly significant. The trend, however, is interesting. Additional cores from arbitrarily selected joints of these two pavements, taken in 1980, showed the same situation to exist as shown in Figure 9. Bashore and Price also found indications that the rate of attack on the bottom of the joints may decrease at least temporarily, for this type of pavement, as age of the pavement increases beyond the first few years. If true, this suggests perhaps that the height and duration of standing water within the joint may be such that the very bottom of the joint is affected more than the materials higher on the joint face. Drainability of the base, therefore, could play an important role, as well as the susceptibility of the coarse aggregate particles in the mix to this type of deterioration.

If water is removed quickly by a highly drainable base layer, the probability of saturating the susceptible aggregates in the bottom of the slab should be lowered to some extent. Open graded base materials also should prevent capillary water in layers below from reaching the bottom of the slab. Recent investigations by the Portland Cement Association (10, 11, 12, 13) have provided some interesting information related to this type of deterioration.

Another factor may be related to the early deterioration of the bottom of the joint face. Several investigators have found that spalling of the bottom of joints or cracks occurs quite early in the life of the pavement, especially it seems, if the slab length is quite long. Although D-cracking is known to start at the bottom and work upward, it seems in some cases that the onset of the first spalling occurs at a date that is quite early for D-cracking to have begun. The fractured surfaces of recent cores from the 22-m (72 ft) slabs at Clare (similar to No. 18 shown in Fig. 9) were examined for evidence of the cause of failure. The spall surfaces appeared to be more probably caused by mechanical interference than by aggregate deterioration. There was no obvious evidence of the numerous small cracks in the aggregates that are typical of D-cracking. If the first spalls are indeed caused by mechanical interference, it would help to explain the early differences between the long slab and short slab pavements, since the greater joint opening of the long slab pavements would increase the probability of base materials being picked up in the joint as it closes. Also, there is the slight effect of downward curling at the joints as the pavements are warmed by the sun on the top surface.

The experimental pavement installation north of Clare was built with a range of drainability that is very rare. The fact that the concrete mix has some weaknesses that are not unusual in many parts of the country, has allowed the effects of the large variation in base drainability to become evident at a relatively early age. Deep scaling of the joint surfaces and bottoms of slabs in the non-draining bituminous base sections has caused the plain pavement to fault. Early stages of D-cracking also are evident on these same sections. Free-draining base sections are performing well.

It has become increasingly apparent in recent years, that water under pavements is a major deleterious factor, regardless of the type of pavement. Base pumping, joint faulting, pavement failures, poor rideability, shoulder distress and concrete durability all are, or can be, related to the occurrence of free water under the pavement. Because of these data, and information from other projects throughout the state that pointed to lack of drainability in the base layer as a problem, the Department has recently issued new standard cross-sections, calling for open graded free draining base materials for use on future Interstate projects.

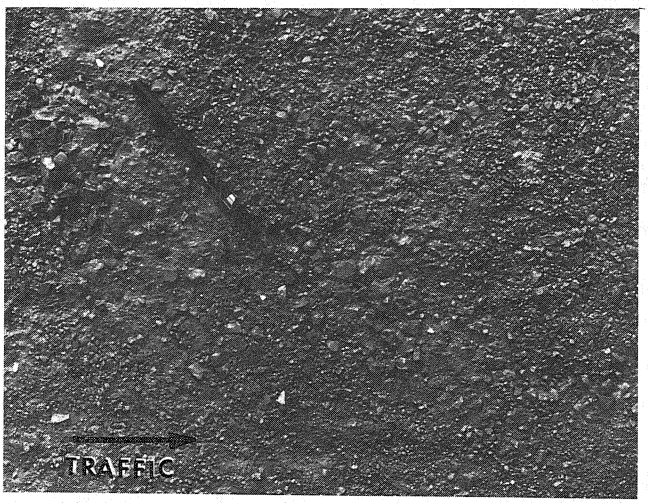
A related but somewhat unusual case of no-load transfer pavement on nondraining base occurred several years ago. In 1965, a very stiff widened base was placed over an old concrete pavement and it was topped with a wider plain concrete pavement without load transfer, evidently assuming that the very strong base would prevent faulting of the new pavement. The new base was constructed of crushed limestone of 20 to 25 cm (8 to 10 in.) nominal thickness, with all the crusher fines left in. The stone was further reduced by the fine grading operation so that all the voids in the base were filled with fines. Shoulders of the same crushed limestone were added. The net effect of the operation was much the same as a nondraining bituminous base, except that the base was highly erodible. The limestone base and shoulders caused ponding of water under the slabs.

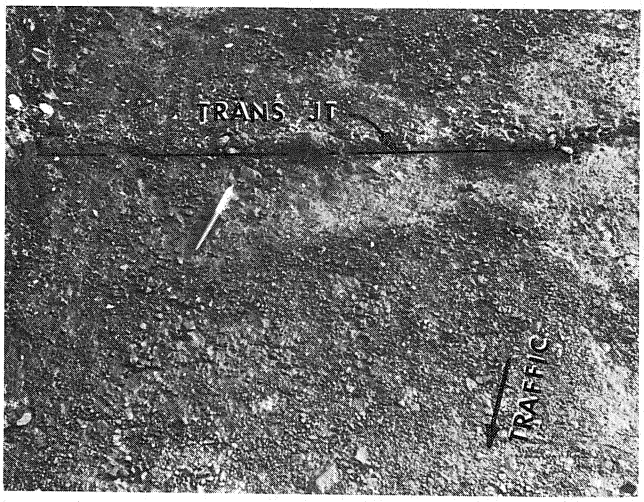
Although truck traffic was very light on this rural road, joint faulting developed early. When slabs were removed, spanning a joint, it was found that water pumped by trucks had caused a void under the 'leave' slab due to the movement of the finer limestone particles back under the approach slab. Some of the so-called 'finer' particles exceeded 2.5 mm (0.1-in.) in diameter, demonstrating the power of the fluid pump that works beneath the pavement. Figure 10 shows the faulted pavement joint, faulted base after pavement removal, and the eroded area beneath the sunken slab. Therefore, one must not conclude that the absence of ultra-fine particles will prevent faulting in cases where water is retained beneath the pavement for long periods of time, and truck traffic is present.

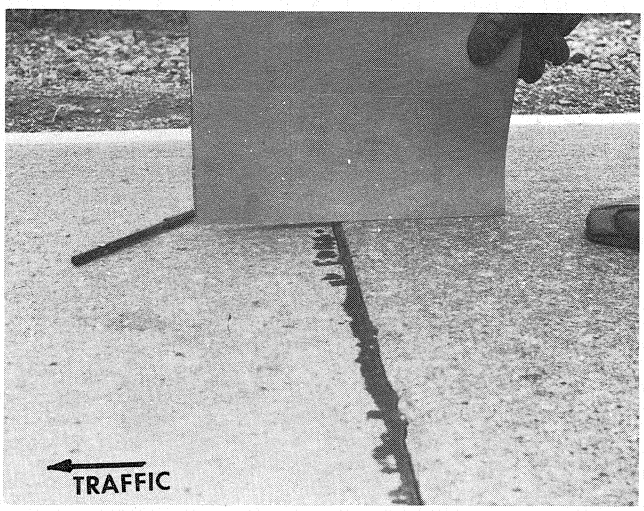
D-Cracking

While Michigan has not been plagued by the extremely fast acting deterioration of aggregates that has occurred in some other states, the figures pre-

Figure 10. Removal of faulted pavement slab from Route 151 revealed the faulted limestone base with a void under the sunken slab. Note the exposed coarse aggregate particles where all finer material has been transported back under the other slab. Particles larger than 2.5 mm (0.1 in.) were removed.



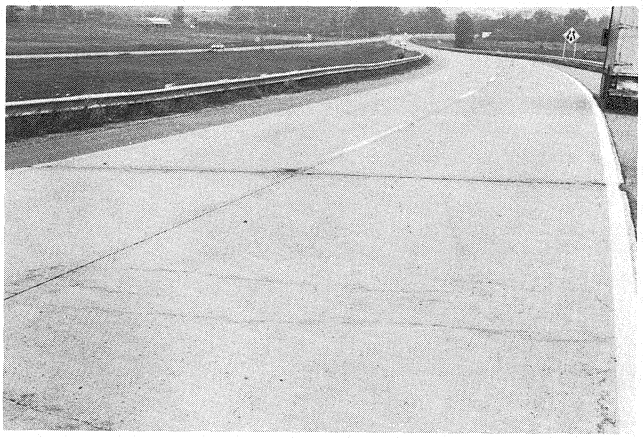




viously shown give an indication of the slower but effective progression of the same general type of concrete pavement deterioration. Many other jointed pavements in the state, constructed from more durable gravel, limestone, and slag aggregates, perform better than the ones shown here. Still others in some localities behave in a more classical D-cracking fashion at a relatively slow rate. The following describes one of these latter cases.

Figure 11 shows a general view of a ramp pavement on US 27 southeast of Mt. Pleasant. This pavement is fairly typical of some of the mainline pavement in the vicinity. Departmental records indicate that it was built early in 1961, and that the locally available course aggregates were known to be susceptible to freeze-thaw deterioration. Those aggregates reportedly were to be improved to some extent by blending a fraction of aggregate from another source. Design life for the pavement was 20 years. Commercial traffic is relatively light. Slab length is 30.2-m (99 ft) with poured joint sealants and base plates.

Figure 11. General view of the US 27 ramp near Mt. Pleasant. Note staining of surfaces near joints and cracks. Pavement was built in 1961, and was 18-1/2 years old when photographed. It has 30.2-m (99 ft) slabs, base plates, and poured sealants. The nearby ramp, shown in the background, contained a premium grade coarse aggregate and shows no D-cracking at the surface.

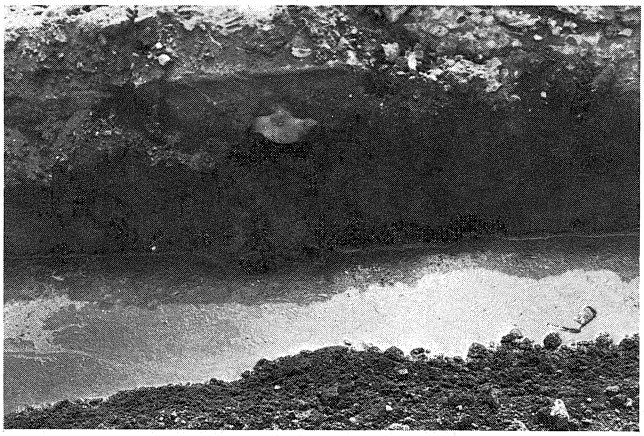


While most pavements in Michigan do not exhibit the traditional D-cracking symptoms shown by portions of this particular project, it does occur at many sites throughout the state. The time frame for crumbling seems to be in the 15 to 20-year range, with the discoloration phase beginning around the 10-year mark. It is apparent, also, that the disintegration of the lower joint faces on many projects throughout the state results from a similar cause, even though the typical symptoms do not show up at the top surface of the pavement. The collection of water in transverse joints, especially those with poor seals and base plates, appears to be a major factor in the type of deterioration that exists.

As the bottom of the joints turn to rubble, the holding capacity for water increases. The dense graded base course and shoulder material used in the state for many years form a barrier to the escape of water, with the result being a continually wet condition within the joint. Disintegration of the top of the joint face adds still more to the water gathering tendency of the joint. Figure 12 shows an

edge view of an older pavement, taken on a pavement-removal project, after the shoulder had been bladed away. Note the miniature mud slide that resulted from water running out of the joint.

Figure 12. Edge view of pavement on demolition job where shoulder material has been bladed away. The small mud slide caused by water running out of the joint marks the joint's location.



The evidence of continuing disintegration in newer pavements, although at a lower rate than on older jobs, indicates the necessity for additional consideration in the selection of materials for the pavements and bases, as well as close attention to joint sealing. Recent changes requiring free draining base materials should aid considerably in reducing the problem on future Interstate projects by keeping the joint areas free of water more of the time. Additional considerations for selection of aggregates for concrete, and for determining maximum size still are required, and now are under study in this laboratory. It is evident from the results to date that improvements have been made but that still more are required.

All of the evidence confirms the fact that has been found by several previous studies; water--in and under the pavement--is a major harmful factor and one that seriously accentuates other weaknesses in the system.

Longitudinal Joints and Lane Ties

Michigan's older pavements had lane ties that were 13 mm (No. 4) rebar at 1 m (40 in.) c to c., with a nominal 3.2-mm (1/8 in.) saw cut and cold-applied sealant. Bulkhead joints had nominal 13-mm (1/2 in.) diameter hook bolts or similar tie devices, and these joints were not sealed. Experience had shown that untied joints opened at early age, and then continued to widen with time. However, with the advent of bare pavement policy, the increase in salt usage caused lane ties to fail on some pavements that were 18 to 20 years of age or older.

Along with other design improvements in recent years, tie bars have been increased to 16 mm (No. 5) and now are required to be epoxy coated. The saw cut on centerline has been increased to 6.4 mm (1/4 in.) width to form a better reservoir for sealants and hot-poured rubber asphalt sealants are specified. Bulkhead joints now require the same joint detail and seal as centerline joints.

Compatibility of Slab Lengths

It has been adequately demonstrated that slabs placed side by side should be tied together so as to prevent separation, to provide mutual support across the joint, and reduce influx of water. However, in many cases where separate functions are performed by the adjoining slabs (such as mainline pavement and shoulder) it has been customary in many places to use different basic designs for the two types of pavement. For instance, consider the use of unreinforced, short-slab shoulders adjacent to jointed reinforced or continuously reinforced mainline pavement.

Once the slabs are in place, expansion and contraction of the different sized pieces cause additional stress concentrations that generally result in additional or wider cracks in the pavement. Increased crack widths in reinforced pavements allow salt solutions to attack the rebar and, in time, will generate failures of the steel. Similarly, when a widening lane is attached to an older pavement where the steel has fractured at mid-slab, a crack will very soon spread through the new slab.

The occurrence of commutative cracking in pavement slabs adjacent to short slab shoulder joints appears to be a secondary deleterious effect, and can be outweighed by the beneficial effects of having a full-depth structural shoulder and sealed longitudinal joint at the edge of the pavement. This requires provision for relative longitudinal motion at the longitudinal joint. The performance of the Illinois I 80 project near Joliet seems to confirm this fact, and although our own experimental section on I 69 is still in quite good condition, the same tendency seems to be developing (14). Figure 13 shows general views of the experimental shoulder job on I 69. Bituminous shoulders have separated from the pavement and settled to varying degrees, providing ready access for water into the base. When one considers the structural effects of the tied concrete shoulder and the safety feature provided by the rumble strip on the concrete, along with ever-increasing law-suits for 'low-shoulders,' the tied, sealed concrete shoulder may have increasing appeal. Petroleum price increases also have brought the costs of the two types of full-depth shoulder closer together.

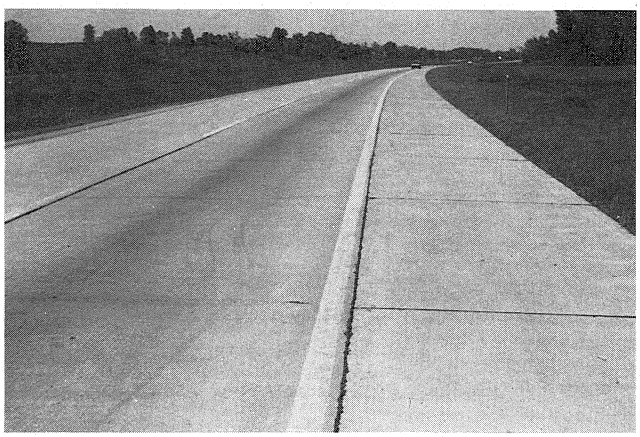
Both practical and technical considerations seem to call for matching the type of design used in the mainline pavement and the shoulder, so that joints correspond, especially when integral construction is permitted.

Load Transfer Dowel Bars

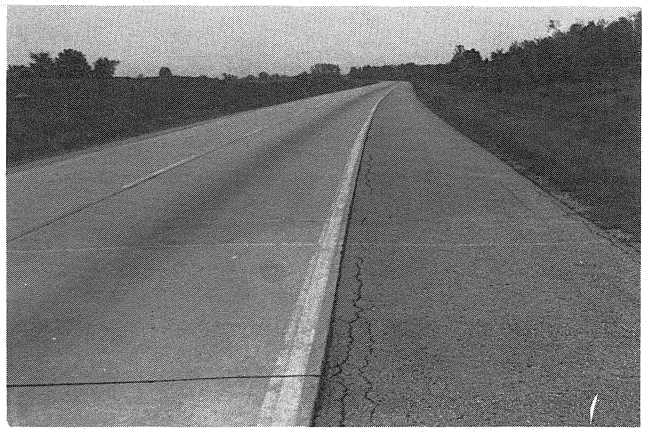
Michigan's extensive experience with load transfer bars has shown them to be effective in the prevention of faulting. However, the older, uncoated dowels tend to corrode near the joint face; and increased pull-out resistance causes failure of the reinforcement at mid-slab in many cases, as the pavements grow old. Corrosion resistant coatings are beneficial in providing longer-lasting dowel performance, and we have had epoxy, polyethylene, nickel, and stainless steel coated bars, in service for from 5 up to 21 years (15). We have concluded that the high density polyethylene coating provides the best combination of low friction, moderate cost, and good corrosion protection. This coating is quite subject to damage in handling and installation and, therefore, requires extra care, but the performance obtained seems to warrant the required precautions.

Figure 13. Experimental concrete and bituminous shoulder installation, I 69 near Olivet. All three bituminous type shoulders have separated from the concrete pavement, and settled to varying degrees, especially near transverse joints. The concrete is in good shape, except for some localized problems with poorly made expansion joints. Communicative cracking of pavement slabs opposite concrete shoulder joints can be seen in many locations on this project, but the cracks still are quite narrow.

Concrete shoulders



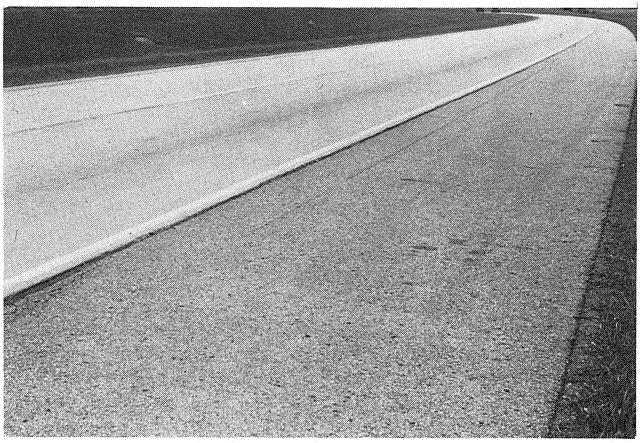
1968 standard bituminous over gravel (170 lb/sq yd)



Full depth bituminous base with seal coat



Full depth bituminous base course and wearing course



Stainless steel coating is effective but costly, and not easily obtainable in reasonably close fit between coating and bar. Epoxy coatings are useful, but do not appear to provide probability of as long-lasting protection. Careful attention to bar preparation and quality control of this coating is required.

Conclusions and Recommendations

1. Water is a primary factor in the deterioration of concrete pavements. Free water in and under the pavement leads to increased deterioration rates and significantly increases the effects of other weaknesses that may exist.
2. Pressure relief and preventive maintenance concepts can be effective in reducing emergency repairs on pavements that have joint deterioration.
3. Corrosion protection should be applied to load transfer dowels. Plastic coated dowels seem to provide a good combination of long term corrosion protection, low pull-out resistance, and moderate cost, though additional care in handling and placement are required. Stainless steel can be effective but is expensive; and if epoxy coatings are used, careful attention should be paid to quality control.
4. The use of open-graded base materials with adequate drains should be encourage. Dry base conditions appear to significantly improve the performance of pavements, and this is especially important when other adverse factors are present.
5. Special attention should be paid to the course aggregates used, with mix changes and limits on maximum size being established when it is necessary to use lower grade materials, so as to limit as far as practical, the adverse effects of these materials on the long-term performance of the pavements.
6. Pavement faulting in no-load transfer pavements results from the pumping of erodible material from under the 'leave' slab back under the 'approach' slab. Free water under the slab and truck traffic are the instruments that move the materials. Erodible materials need not be ultra-fine particles, as crushed stone pieces more than 2.5 mm (0.1 in.) diameter were found to have been moved by such action. Providing drainage seems to be the most reasonable means to combat the problem, since

it is apparent that significant quantities of water penetrate the system.

7. Improved joint seals seem to be warranted, both in transverse and longitudinal joints, to eliminate incompressible materials, minimize water infiltration, to minimize also the influx of fine materials that would tend to plug the drainage channels in the base, and to keep salt laden water away from embedded steel.

8. D-cracking and/or closely related phenomena are present in many of Michigan's older pavements. While it does not appear at such an early age as in some other states, the problem does exist and must be adequately addressed to improve pavement performance over the long term.

9. Full-depth, tied, sealed concrete shoulders offer benefits in safety and apparently improve the performance of the mainline pavement by improving lateral surface drainage and offering structural support. If shoulders and pavement slabs are integrally cast, compatible design is recommended, matching reinforcement and slab length. If separately cast, matching design is still desirable, but may be omitted with secondary deleterious effects, if effective bond breakers are used between the shoulder and pavement.

References

1. "Recommendations Concerning Concrete Pavement Design and Maintenance," Michigan DOT, Research Report No. R-790R, November 1971.
2. Chiunti, M. A., "Experimental Short Slab Pavements, Construction Report," Michigan DOT, Research Report No. R-1016, August 1976.
3. Green, T. M. and Novak, E. C., Jr., "Drainage and Foundation Studies for an Experimental Short Slab Pavement," Michigan DOT, Research Report No. R-1014, February 1977.
4. Arnold, C. J., "Pressure Induced Failures in Jointed Concrete Pavements and a Machine for Installation of Pressure Relief Joints," Michigan DOT, Research Report No. R-949, December 1974.
5. Simonsen, J. E., "Repair at Concrete Pavement Joints by Precast Slabs," Michigan DOT, Research Report Nos. R-762, R-804, R-822 and R-869.
6. Simonsen, J. E., "Development of Procedures for Replacing Joints in Concrete Pavements," Michigan DOT, Research Report Nos. R-968 and R-1020.
7. Simonsen, J. E., "Comparative Field Study of Joint Repair Techniques," Michigan DOT, Research Report Nos. R-859 and R-1063.
8. Oehler, L. T., "Survey of Pavement Joint Conditions," Michigan DOT, Research Report No. R-789, October 1971.
9. Bashore, F. J. and Price, A. W., "Effectiveness of Neoprene Seals in Preventing Concrete Pavement Contraction Joint Deterioration," Michigan DOT, Research Report No. R-1122, July 1979.
10. Stark, D. and Klieger, P., "Effect of Maximum Size of Coarse Aggregate on D-Cracking in Concrete Pavements," Portland Cement Association.
11. Stark, D., "Field and Laboratory Studies of the Effect of Subbase Types on the Development of D-Cracking," (RD021.01P), Portland Cement Association, 1974.
12. Stark, D., "Characteristics and Utilization of Coarse Aggregates Associated with D-Cracking," (RD047.01P), Portland Cement Association, 1976.
13. Klieger, P., Stark, D. and Teske, W., "The Influence of Environment and Materials on D-Cracking," (OHIO-DOT-06-78), Portland Cement Association, October 1978.
14. Chiunti, M. A., "Experimental Concrete and Bituminous Shoulders (Progress Report)," Michigan DOT, Research Report No. R-898, February 1974.
15. Arnold, C. J., "Performance of Several Types of Corrosion Resistant Load Transfer Bars for as Much as 21 Years of Service in Concrete Pavements," Michigan DOT, Research Report No. R-1151, 1980.