JOINTED CONCRETE PAVEMENTS
DESIGN, PERFORMANCE AND REPAIR

MICHIGAN
DEPARTMENT OF TRANSPORTATION

TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION

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Summary

Background information is presented 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 and still use load transfer and reinforcement. An experimental installation having extreme variations in drainability is discussed and the effects of base drainage on the performance of the concrete pavement as well as the inter-relationships with aggregate quality are demonstrated. Highly variable performance with changes in coarse aggregate source is shown as well. Pavement joints faulting due to rearrangement of fine base materials is shown. 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 pressure relief joints and installing joint filler. A rating system is presented for evaluation of joint condition, selection of joints for replacement, and determination of the rates of deterioration of various sections of roadway. 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.

Conclusions - 1) Free water in and under the pavement leads to increased deterioration rates and 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. 4) Open graded base materials with adequate drains should be encouraged. 5) Special attention should be paid to the coarse aggregate used, with mix changes and limits on maximum size being established when it is necessary to use lower grade materials. 6) Pavement faulting in no-load transfer pavements results from the pumping of erodible materials from under the 'leave' slab back under the 'approach' slab; free water beneath the slab and truck traffic being the instruments that move the material. 7) Improved joint seals appear to be warranted. 8) D-cracking and kindred phenomena are present in many older pavements and should be studied. 9) Full-depth, tied, sealed concrete shoulders offer benefits in safety and seem to improve the performance of mainline pavement by offering improved drainage and structural support.

Background Information

Postwar portland cement concrete jointed pavements on State trunk-lines in Michigan were mesh reinforced, with 99-ft slab length and dowel bars for load transfer. Most were 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 70 ft and base plates were eliminated.

In 1971, the Research Laboratory recommended a preventive mainte-nance concept for the old 99-ft slab pavements (1). During the 1970's, slab lengths on new pavements were reduced still further, to approximately 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 using an un-usually large variation of drainability in the base, ranging from zero to several thousand feet per day. Three sections each of the following pave-ment types were built (all sections are 9 in. thick; all shoulders are full-depth bituminous; the subgrade is sand): A) Standard pavement, reinforced, with 71 ft-2 in. slab length and load transfer, placed on 4 in. aggregate base course over sand subbase. B) Experimental pavement, short slab plain concrete, with skewed joints, variable slab length, and no load trans-fer, placed on a 4 in. base of open graded asphalt treated porous material (ATPM) over sand subbase. C) Experimental pavement, short slab plain

concrete, variably spaced joints at 90 degrees to centerline, with load transfer, placed on 4 in. aggregate base course over sand subbase. D) Experimental pavement, short slab plain concrete with skewed joints, variable slab length and no-load transfer placed on a 4 in. non-draining bituminous base course over sand subbase. There are some problems developing in the non-draining black base sections. Performance of these experimental pavements over the first five years is covered later in this report. The locations of all the various projects discussed in the report are shown in Figure 1.

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 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 (No. 5 rebar) lane tie bars with epoxy coatings have been added. Therefore, the main item requiring further attention for concrete pavements is the quality of aggregates. This item is the subject of further research and evaluation 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 have been 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 2. 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. 3). 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 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 in pavements to prevent blow-ups.
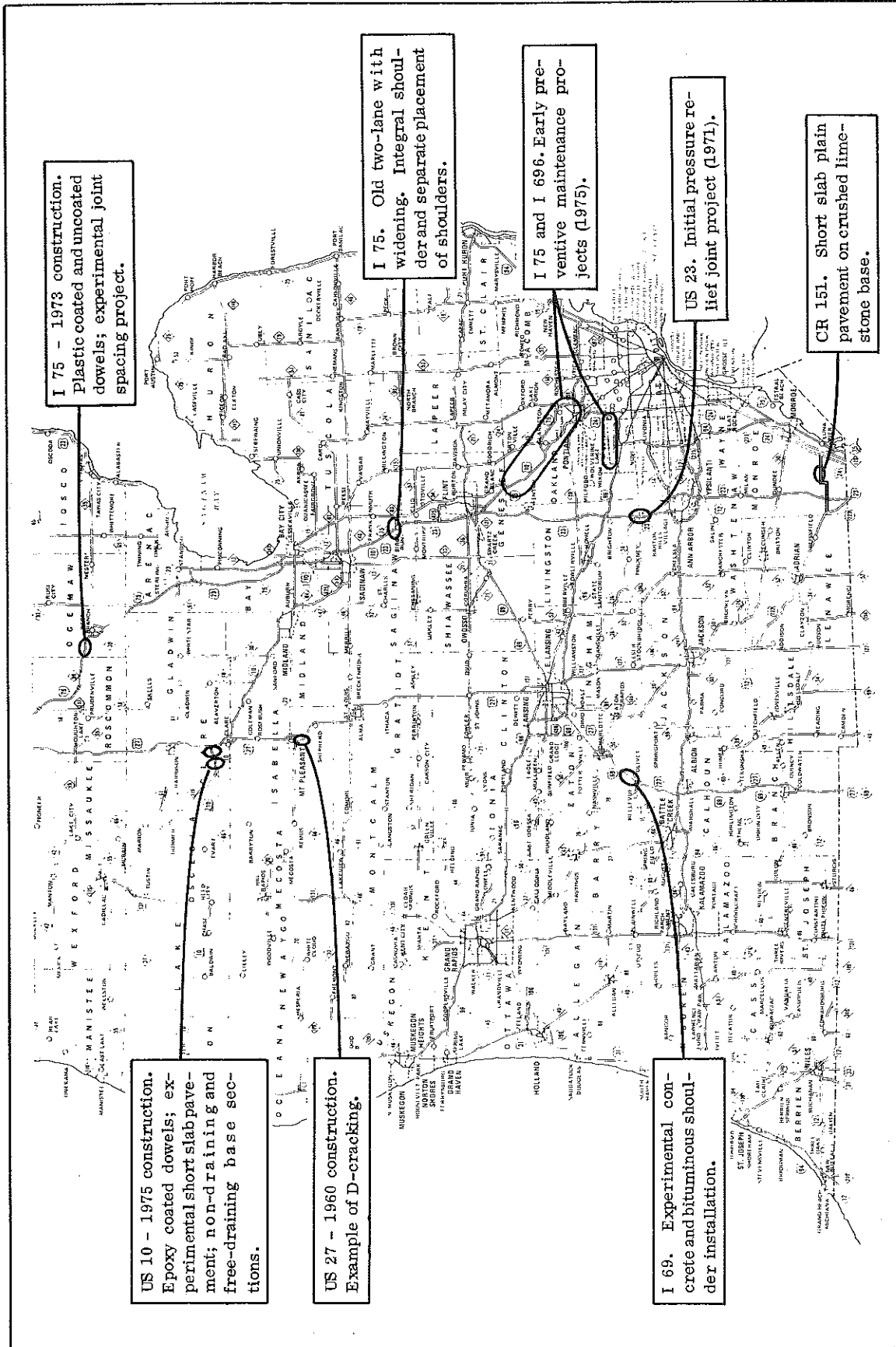
I 75 – 1973 construction. Plastic coated and uncoated dowels; experimental joint spacing project.

I 75. Old two-lane with widening. Integral shoulder and separate placement of shoulders.

I 75 and I 696. Early preventive maintenance projects (1975).

US 23. Initial pressure relief joint project (1971).

CR 151. Short slab plain pavement on crushed limestone base.

US 10 – 1975 construction. Epoxy coated dowels; experimental short slab pavement; non-draining and free-draining base sections.

US 27 – 1960 construction. Example of D-cracking.

I 69. Experimental concrete and bituminous shoulder installation.

Figure 1. Locations of the various projects discussed in the report.
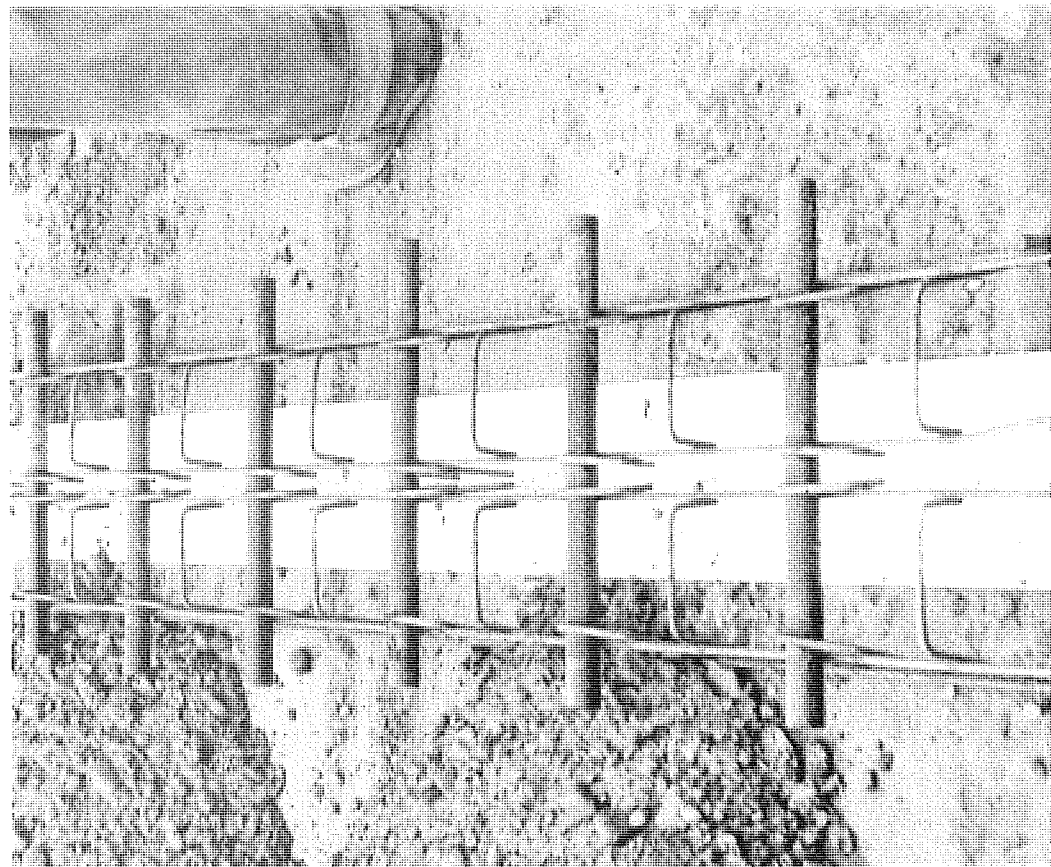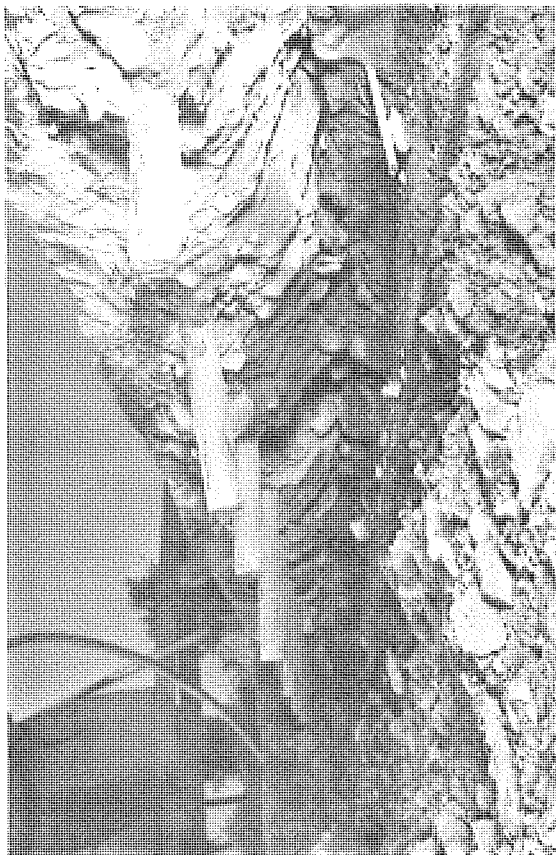
- 4 -

Figure 2. Base plates in new construction (left) were placed to keep aggregates out of the joint and to provide support for the load transfer basket during construction. The photos at right, however, show the long-term effects of increased joint face deterioration that resulted when salt and moisture were trapped in the joint.
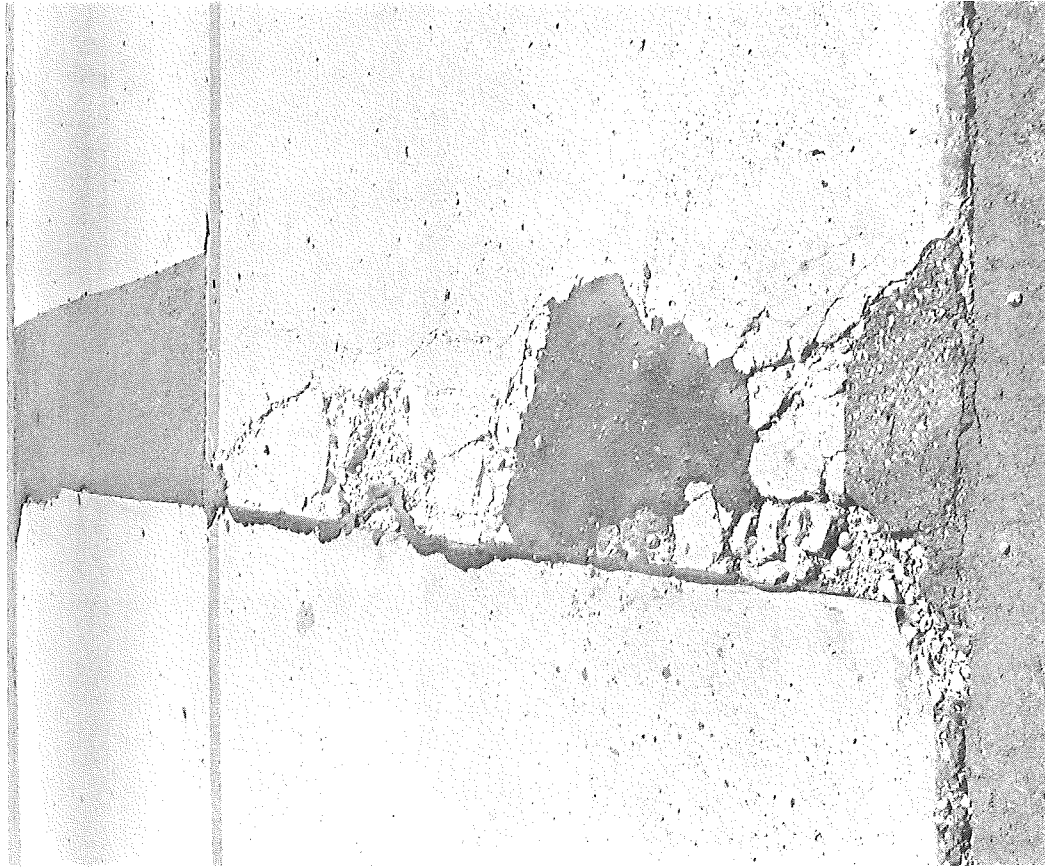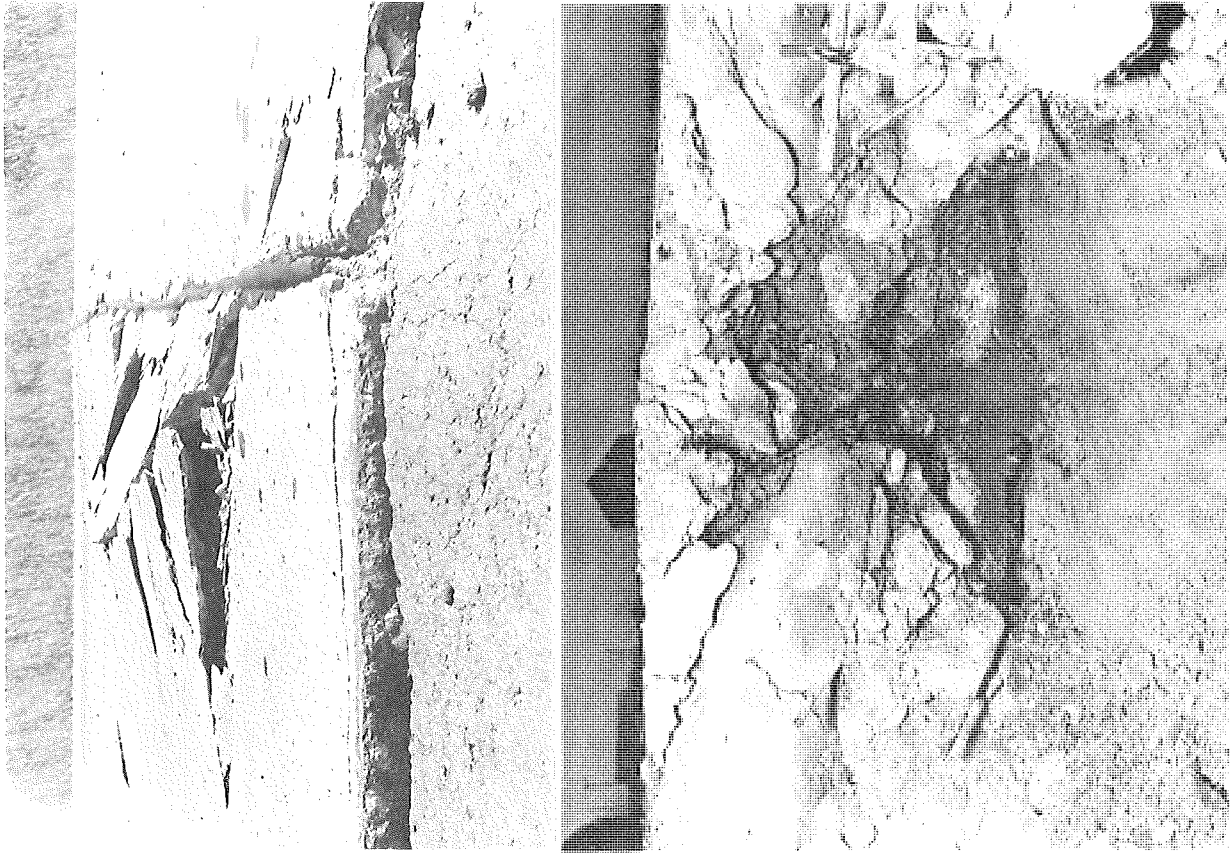
Figure 3. Joint crushing (left) or sometimes severe blow-ups (right) developed at the deteriorated joints when hot spring and summer weather and moisture caused expansion of the pavement slabs.

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 99 ft by 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 Division 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 8,800 ft of pavement, with spacing varied from 400 to 1,000 ft. An adjacent roadway at the same site, in similar condition, was used as a control section. Instrumentation for joint width measurements was installed and pavement ratings were made. Repeated surveys made over the life 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 4. 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 pavements with preformed compressive 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, and a bituminous mastic cement was used. These joint fillers, and other such joints placed by Departmental forces, performed satisfactorily and did not float out of the joints. However, on some later projects placed by contractors, the fillers floated out of the joints during cool, rainy weather when slabs contracted. Survey
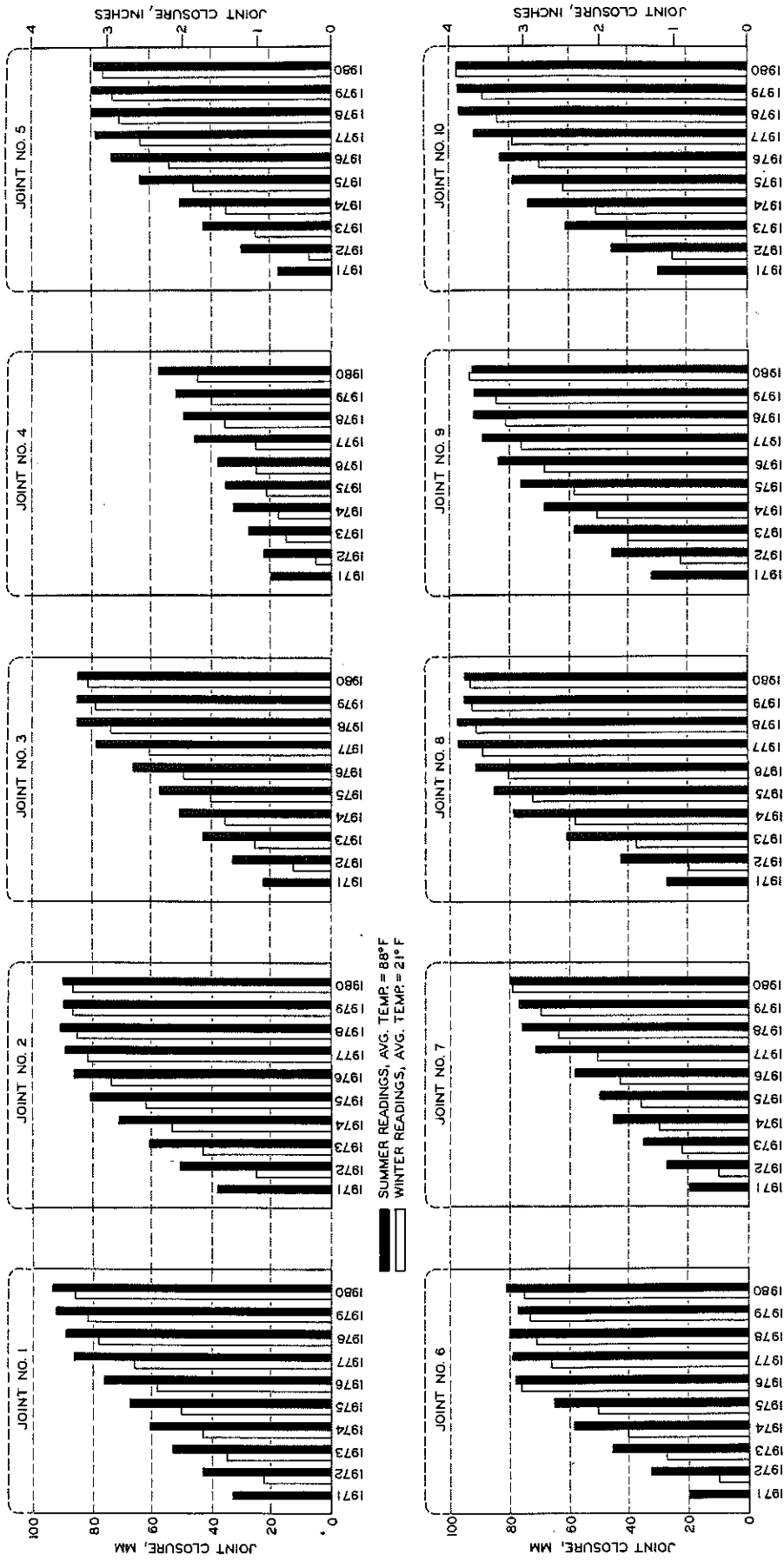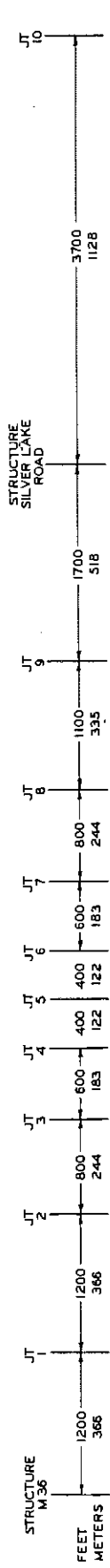
Figure 4. Location of pressure relief joints and total closure (US 23).

information shows roughly one out of four ethafoam fillers have been ejected on 45 miles of freeway that were repaired and pressure relieved in 1975.

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 to place the 4-in. filler in 3-1/4-in. joints which attached to the front of a snowplow truck (Fig. 5). That machine has been revised to handle the shaped urethane foam now in use. 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.

Figure 5. Ethafoam pressure relief joint installation device, mounted on snowplow lift, on heavy duty maintenance vehicle. Punch approaching filler (right).
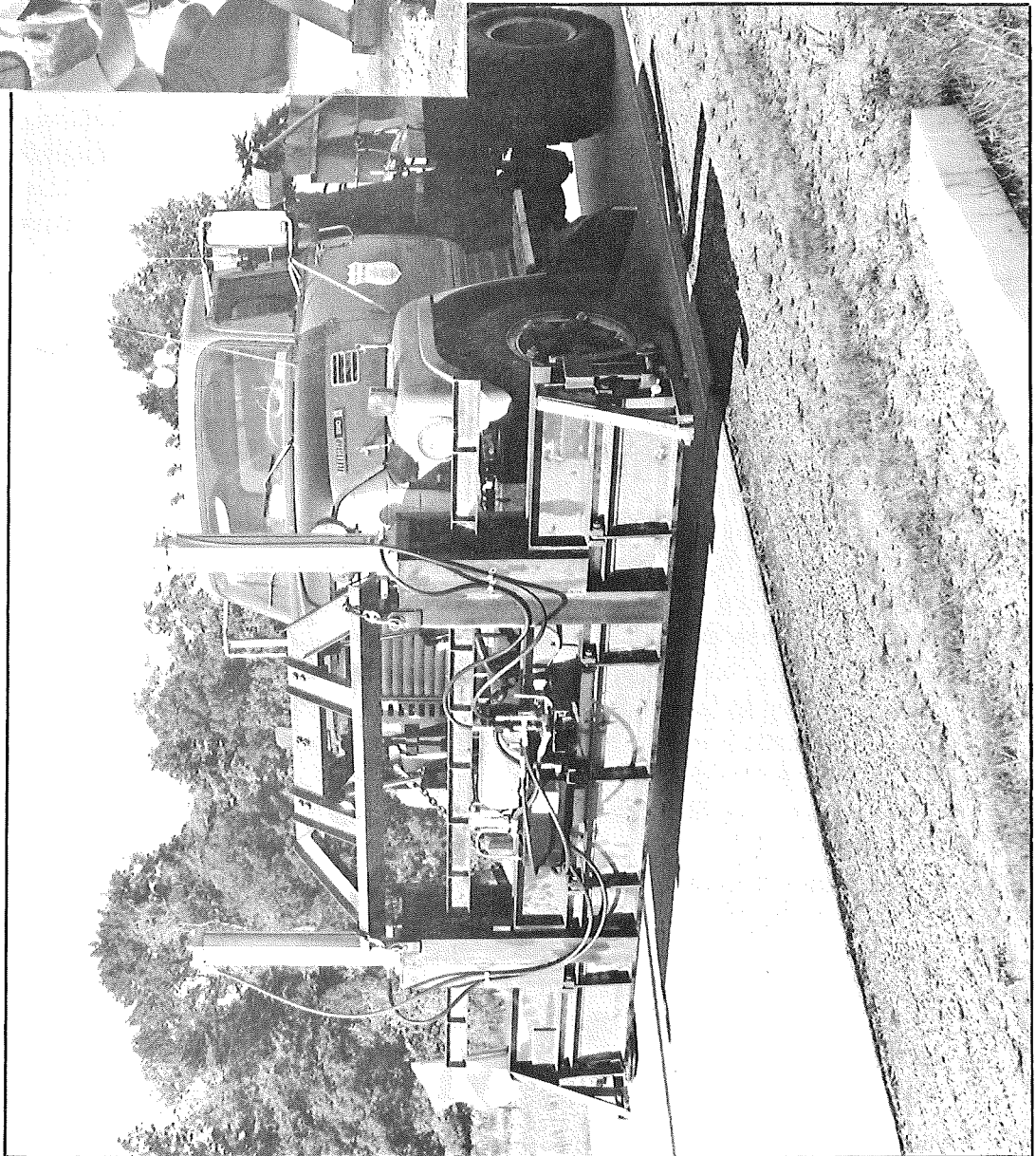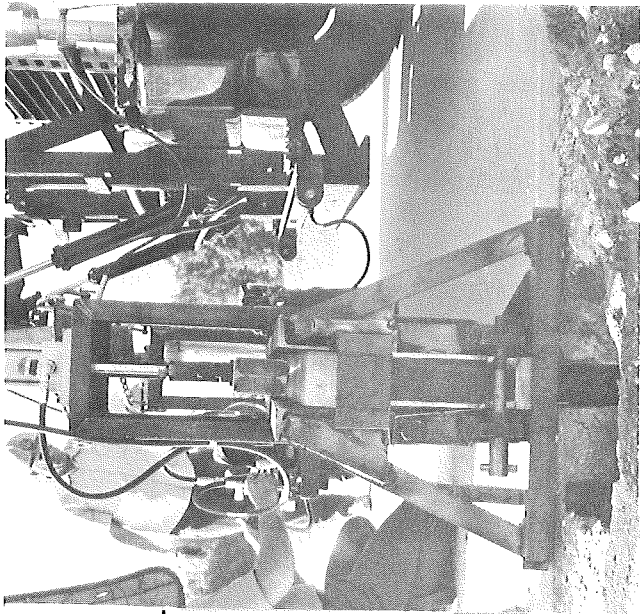
Please note that in all cases here, we are discussing repairs of the older 99-ft slab length pavements with poured sealants and base plates. Experimental work on these projects resulted in the refinement of an existing rating system (see Appendix) so that joint conditions could be quantified, and warrants for joint replacement or pressure relief could be established.

Various joint details were developed for use with repairs, so that contraction, 1-in. and 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 4 in. but not more than 6 in. of expansion space were provided every 850 ft. Joints are rated by estimating the amount of spalling exceeding 4 in. width, plus the amount of corner breaks, in lineal feet along the joint. (The methods of surveying and rating are shown in the Appendix.) Warrants for replacement then were established. Usually 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 the full width of the pavement were made without expansion space. A minimum of 6 ft of pavement removal parallel to centerline is required.

Although methods for re-establishing 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 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 suit-
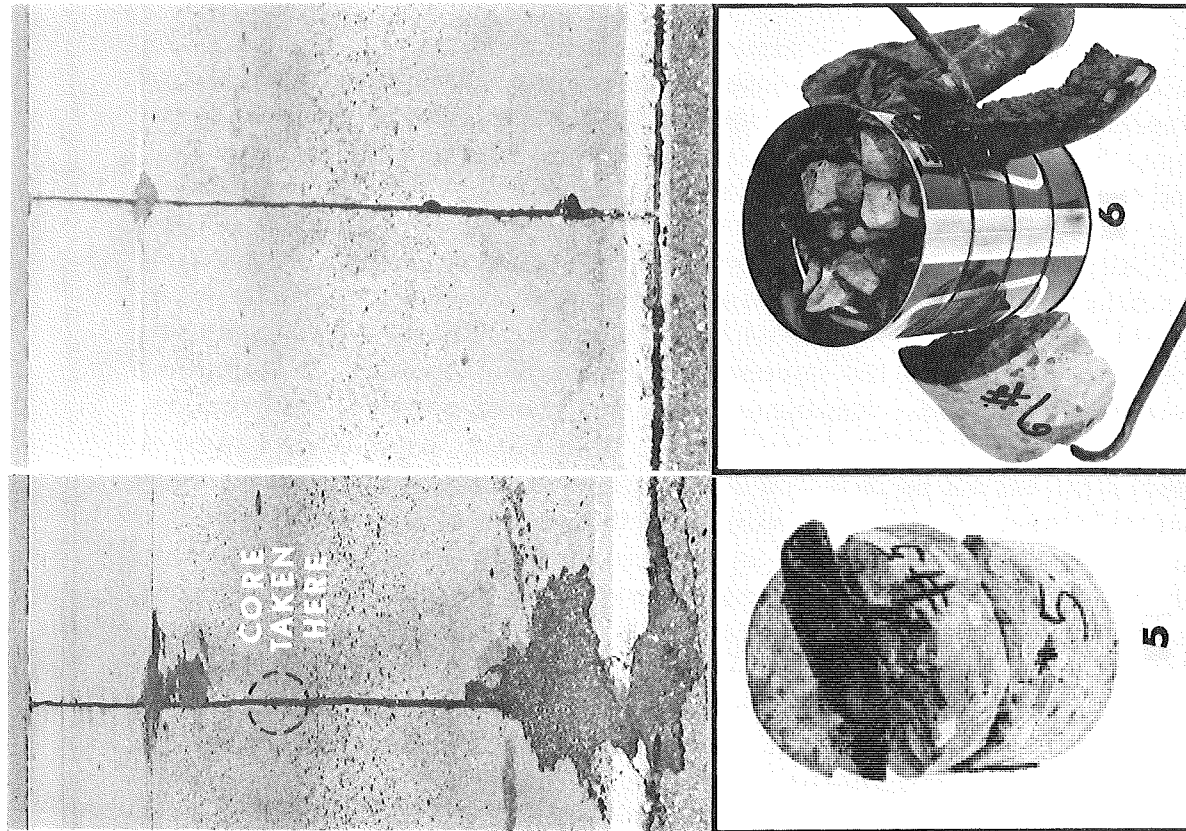
Figure 7. Joints and cores from I 96 near Portland. These are contraction joints (cores shown upside down), constructed 1958, 99-ft slab with base plates. After 21-1/2 years' service on this rural Interstate route the joint faces are badly deteriorated and are fairly typical of pavements of this design and age.
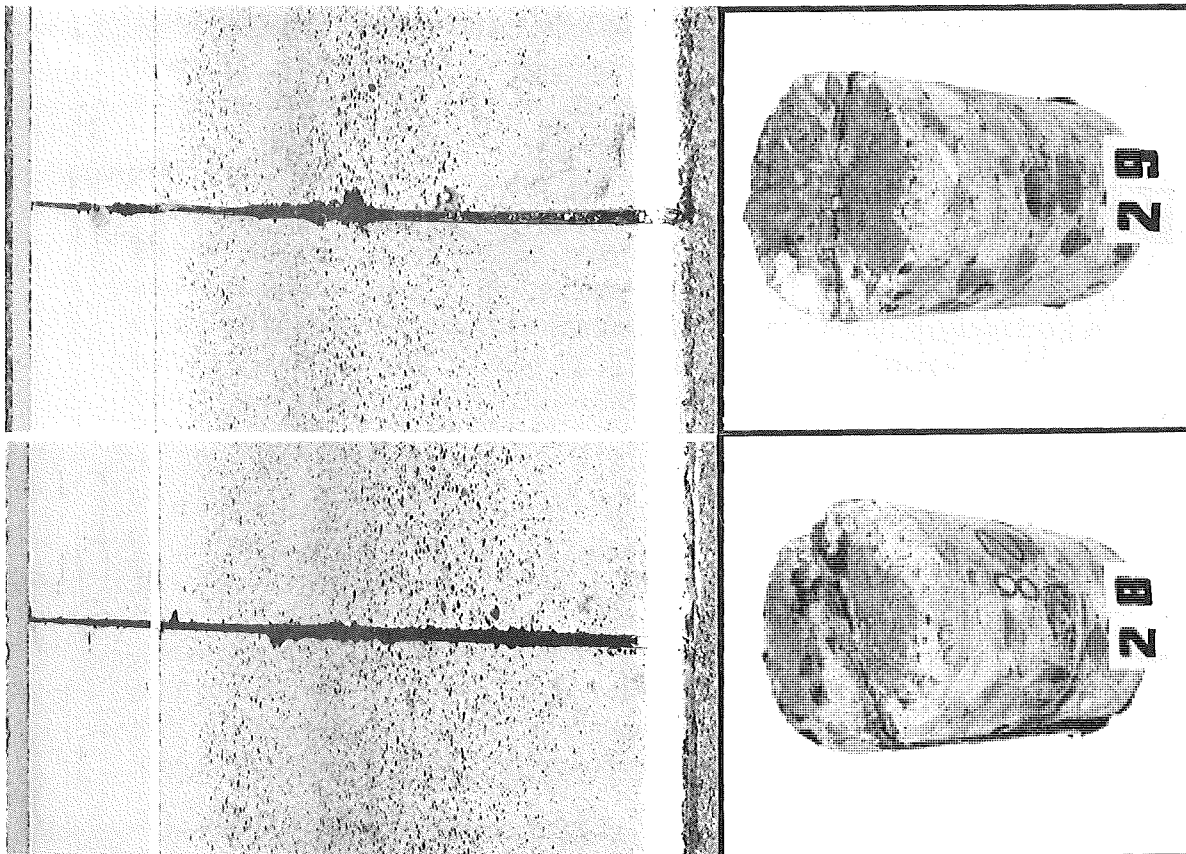

Figure 6. Typical joints and cores from I 96 near Portland. Expansion joints, cores (shown upside down) are typical of the condition of expansion joints in the area. Constructed in 1958, 50-ft slab with base plates, as a relief section for continuously reinforced pavement. After 21 years' service on this rural Interstate route the concrete is in good condition full depth.

able for pavement concrete, the materials readily available in some parts of the State are subject to progressive deterioration. Figures 6 through 11 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 cores in Figure 6 were removed from expansion joints 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 7 shows cores from contraction joints nearby the expansion joints of Figure 6, 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. In 1971, L. T. Oehler reported the results of a limited survey that showed the decreases in rate of deterioration that resulted from improved joint seals and elimination of base plates (8). 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 transverse 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 8 through 11 show deterioration of varying degrees at the bottoms of joints in some of the more modern projects. All of these pavements were placed on 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 core Nos. 16 and 17 in Figure 11 from short slab pavements on the same project as the longer slab pavement core Nos. 18 and 19 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 11. 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 fracture surfaces of recent cores from the 72-ft slabs at Clare (similar to No. 18 shown in Fig. 11) 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 sealing 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.
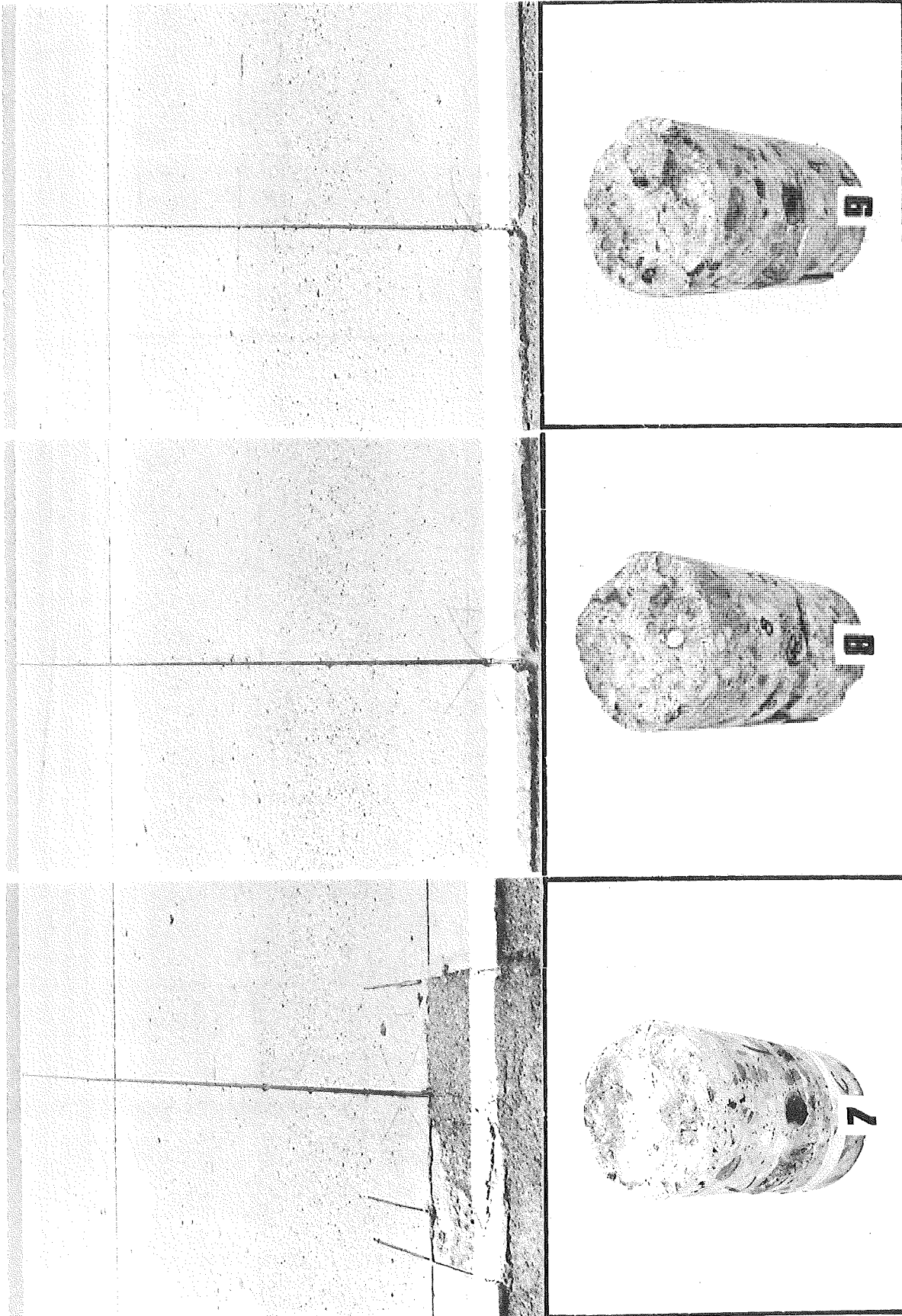
Figure 8. Typical joints and cores from M 52 near Owosso. Contraction joints (cores shown upside down) constructed 1969, 71-ft slabs, no base plate, neoprene seal. After 10 years' service on this rural two-lane trunkline, minor deterioration is seen at bottom slab and horizontal cracks at bar level.

Figure 9. Contraction joints and cores (shown upside down) from I 75 near West Branch. Constructed 1973, specimens 22, 23, 25 and 26 are from 71-ft slabs, specimen 24 from a 57-ft slab, and specimen 27 from a 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 10. Contraction joints and cores (shown upside down) from US 10 north of Clare. Constructed 1974 (but only in service four years when sampled), 71-ft slab, no base plates, preformed neoprene seals. These rural freeway samples displayed moderate concrete deterioration at the bottom of the joint.

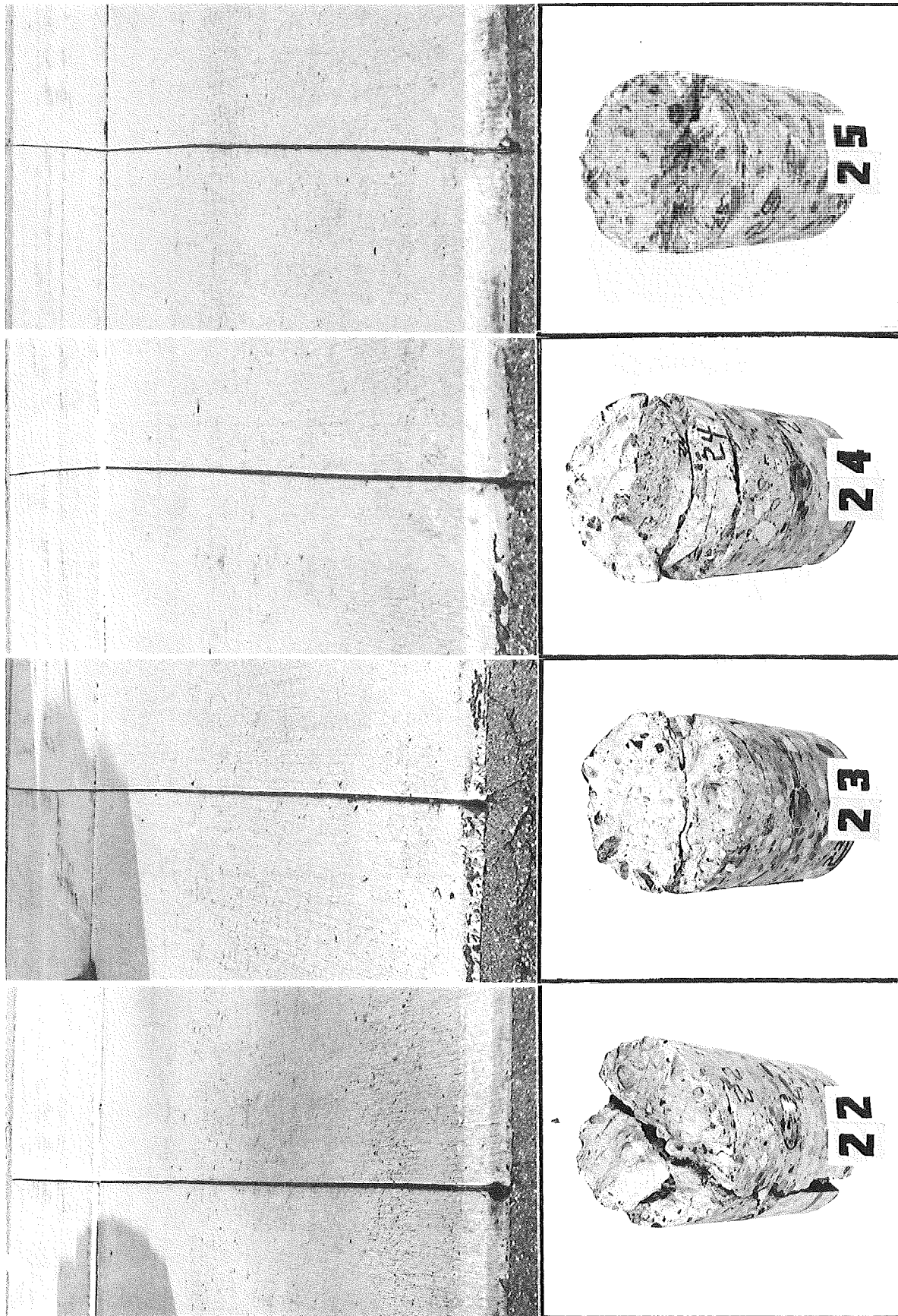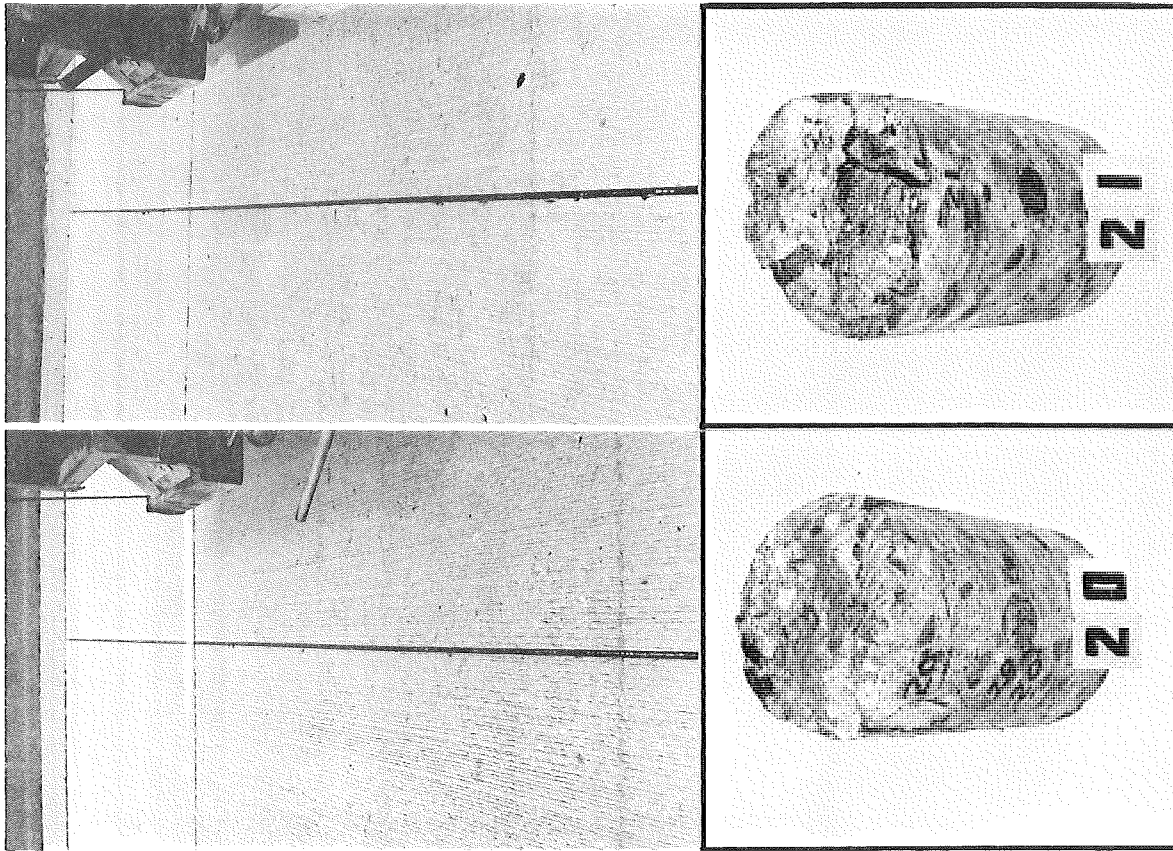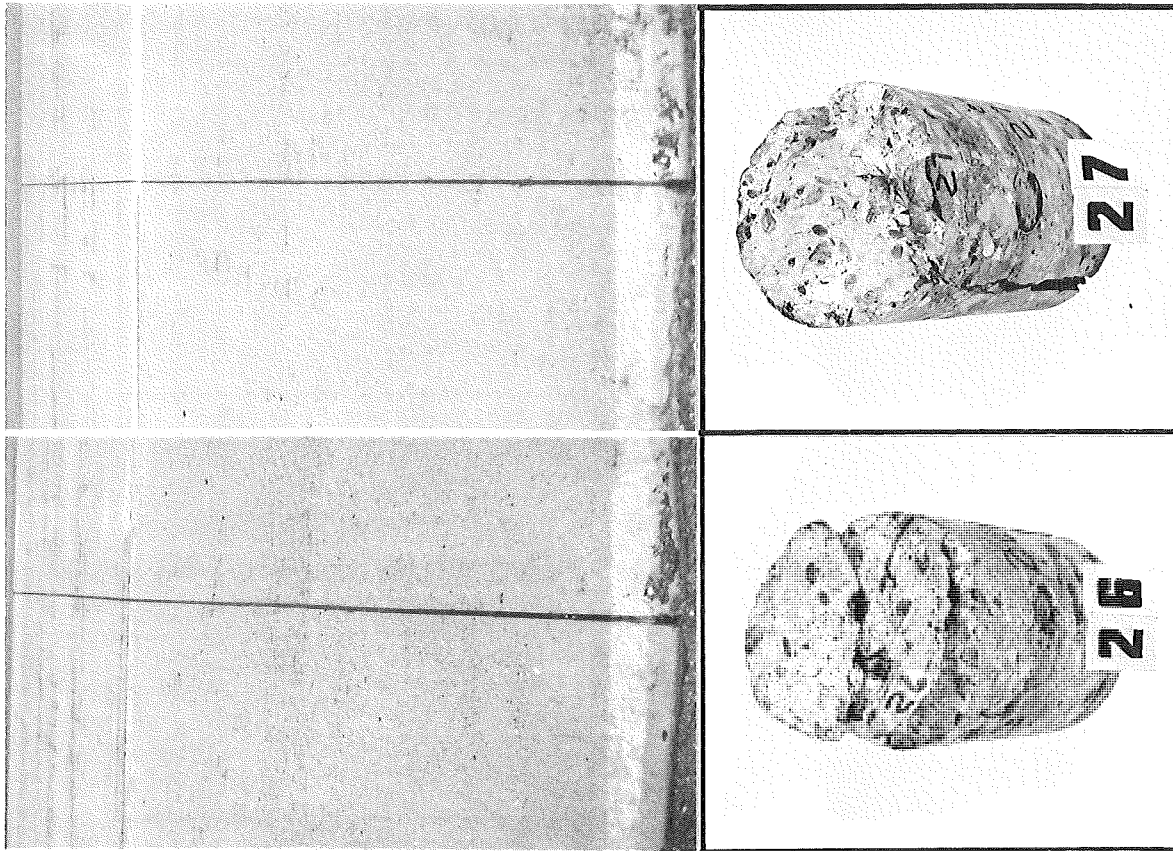Figure 9 (Cont.). Contraction joints and cores (shown upside down) from I 75 near West Branch. Constructed 1973, specimens 22, 23, 25 and 26 are from 71-ft slabs, specimen 24 from a 57-ft slab, and specimen 27 from a 43-ft slab, with no base plates and neoprene seals. This rural freeway had been in service for six years when sampled; all show moderately severe deterioration at the bottom of the joints.

- 17 -

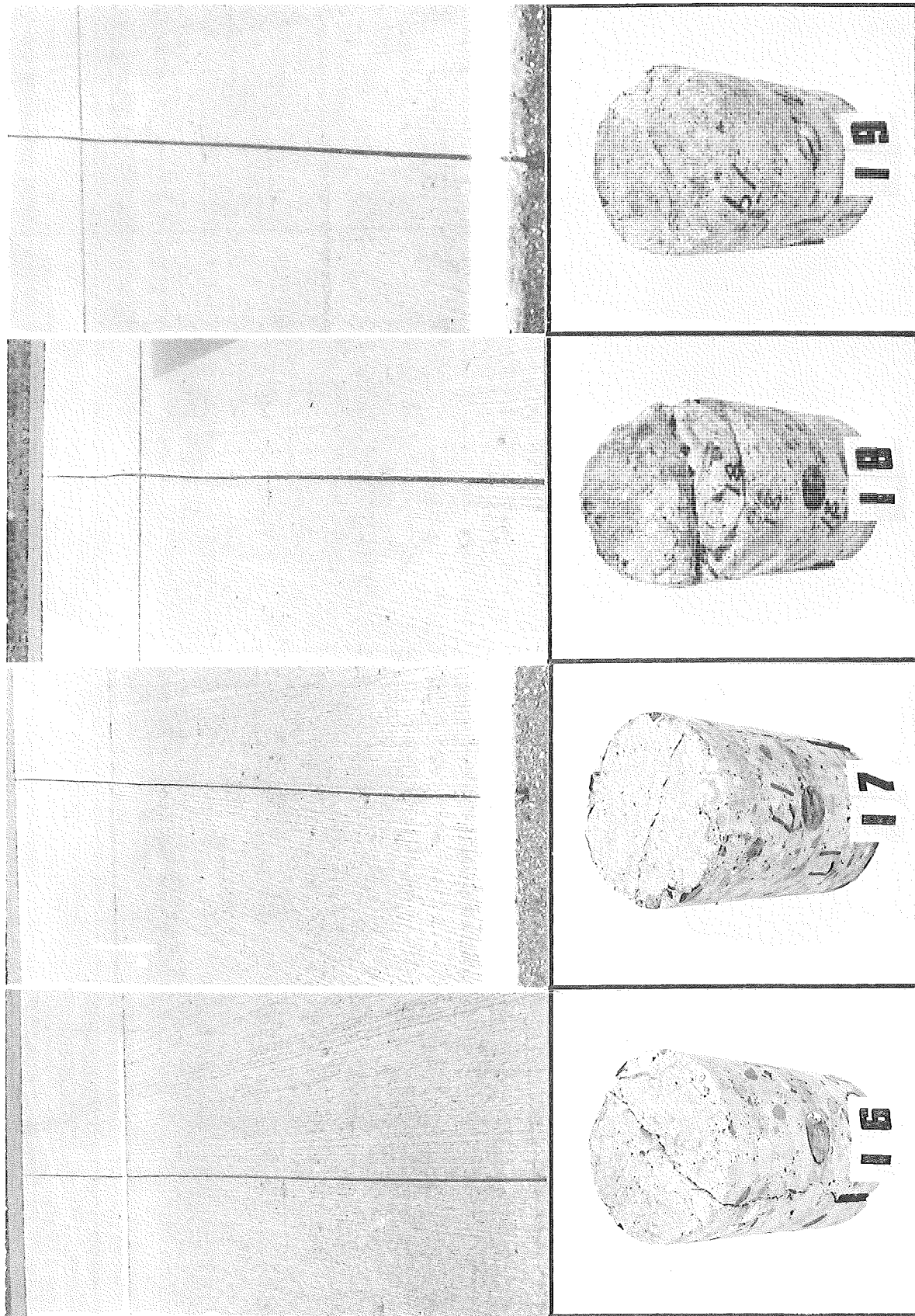Figure 11. Contraction joints and cores (shown upside down) from the gravel base sections of the experimental installation on US 10 north of Clare. Constructed 1975 (in service four years), specimens 16 and 17 from 13 to 17-ft slabs, specimens 18 and 19 from 71-ft slabs of the control section, no base plates, with neoprene seals. Note the much greater concrete deterioration in the core from the longer slab from this rural freeway.

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 projects.

A related but somewhat unusual case of no-load transfer pavement on non-draining base occurred several years ago. An old concrete pavement on route 151 in southeastern Michigan was obsolete because of its narrow width. In 1965, a very stiff widened base was placed over the old pavement and topped with a wider plain concrete pavement without load transfer. Since this was prior to the definitive work on the causes of faulting by the California experimentors, it was assumed, evidently, that the very strong base would prevent faulting of the new pavement. The new base was constructed of crushed limestone of 8 to 10 in. nominal thickness, with all 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 non-draining 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 0.1 in. in diameter, demonstrating the power of the fluid pump that works beneath the pavement. Figure 12 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 ultrafine particles will prevent faulting in cases where water is retained beneath the pavement for long periods of time, and truck traffic is present.

Figure 13 shows portions of Rapid Travel Profilometer traces made in 1972 and again at the same location in 1979. Notice that the size of the faults has continued to increase as the pavement age increases, and at the present level have had a significant effect on the rideability of the pavement, especially for stiffly suspended vehicles. Portions of the 'wartime' plain concrete pavements on the old Detroit Industrial Expressway, built up joint

Figure 12. 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 0.1 in. were moved.

Traffic direction →

Traffic direction →

Traffic direction →
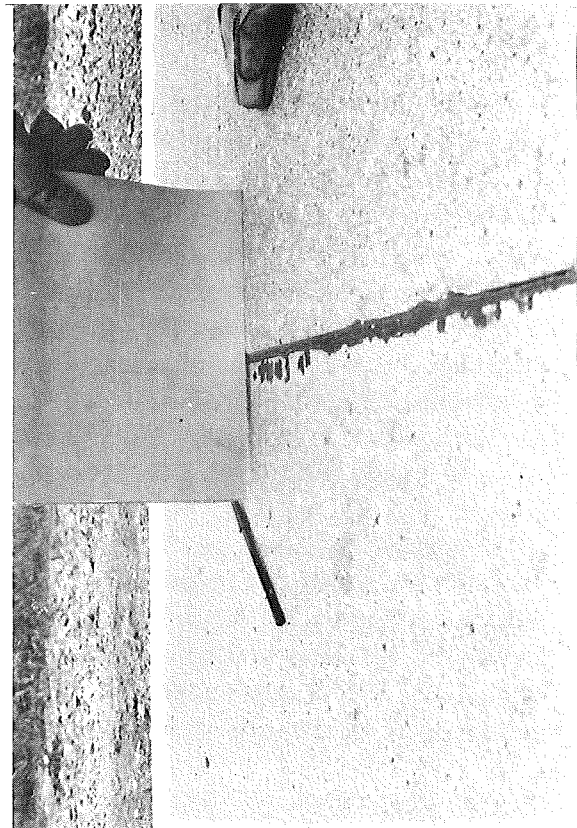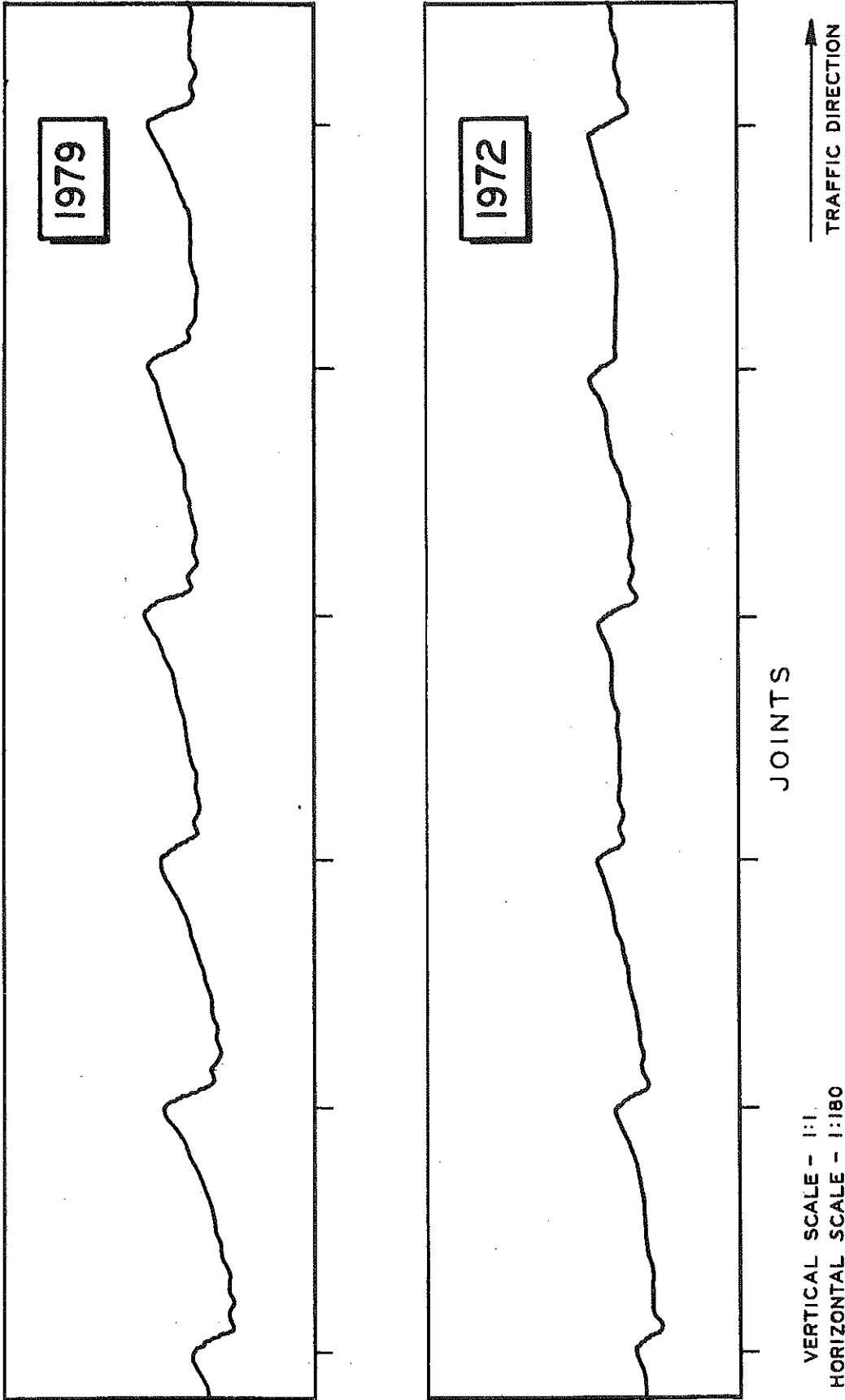
Figure 13. Rapid Travel Profilometer traces showing increase in faulting of short slabs, plain concrete pavement after seven additional years of light traffic.
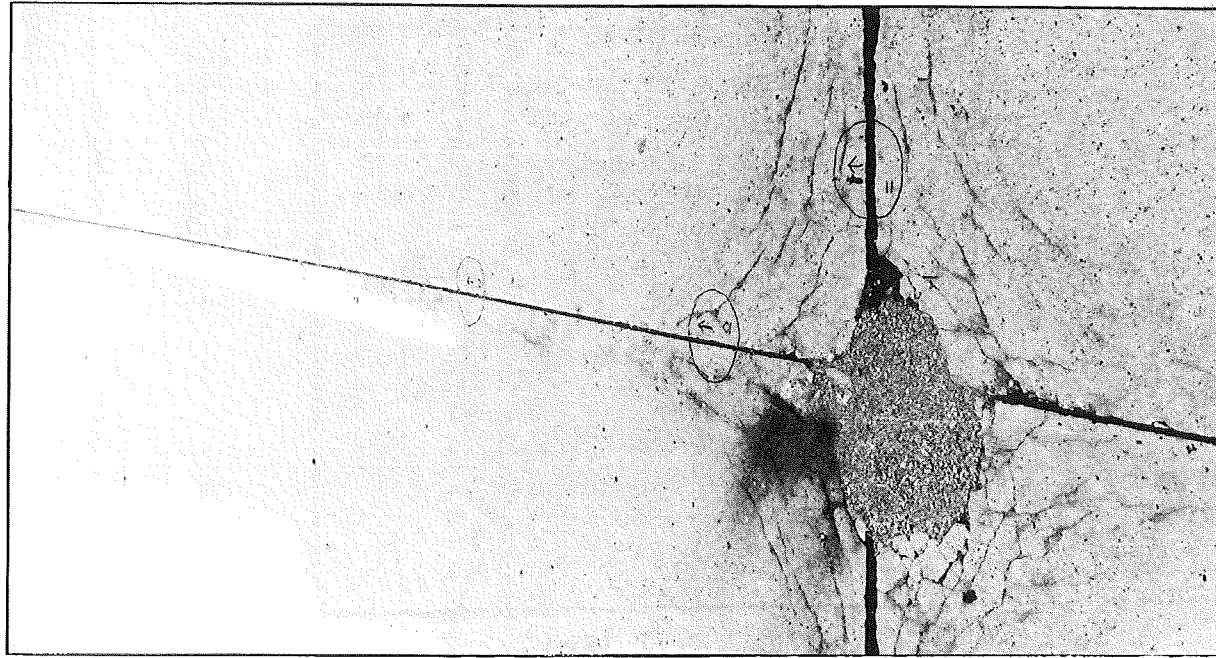
Figure 14. General view of the US 27 ramp near Mt. Pleasant. Note staining of surfaces near joints and cracks. Pavement was built in 1961, and was 18-1/2 years old when photographed. It has 99-ft slabs, base plates, and poured sealants. A close-up view of the joint is also shown. The nearby ramp, shown in the background, contained a premium grade coarse aggregate and shows no D-cracking at the surface.

- 22 -

faults of more than an inch under the heavy truck traffic before they were removed from service. Obviously, the CR 151 pavement would be in far worse condition than it is if significant volumes of commercial traffic were 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 previously 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.

TABLE 1
GRADING REQUIREMENTS FOR COARSE AGGREGATE

Spec. Year	Location	No. and Class	Total Percent Passing Sieves						
			2-1/2 in.	2 in.	1-1/2 in.	1 in.	1/2 in.	3/8 in.	No. 4
1960	Mt. Pleasant	4A	100	95-100	65-90	10-40	0-20	0-5	
1960	Mt. Pleasant	10A			100	95-100	35-65		0-8
Present	Clare	6A			100	95-100	30-60		0-8

Figure 14 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 coarse aggregates were known to be susceptible to freeze-thaw deterioration. Specifications of that time required equal parts of 4A and 10A coarse aggregate (gradations given in Table 1). Design life of the pavement was 20 years. Commercial traffic is relatively light. Slab length is 99 ft with poured joint sealants and base plates. The adjacent ramp shown in the background of Figure 14 was built the previous fall with a premium quality coarse aggregate in the 4A fraction and is in excellent condition after 20 years of service. The only difference between the good and poor performing pavements was that the 4A fraction only of the coarse aggregate in the good pavement was dolomite, while another source was used for the 4A in the poorer example. The 10A and fine aggregates were the same for both the good and bad sections. This project and subject are covered more extensively in a recent research report (14).
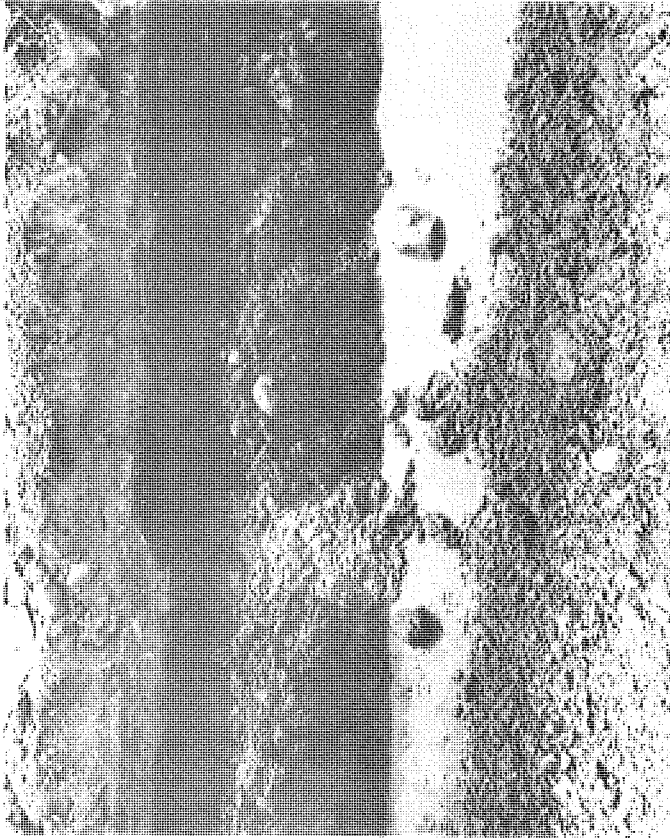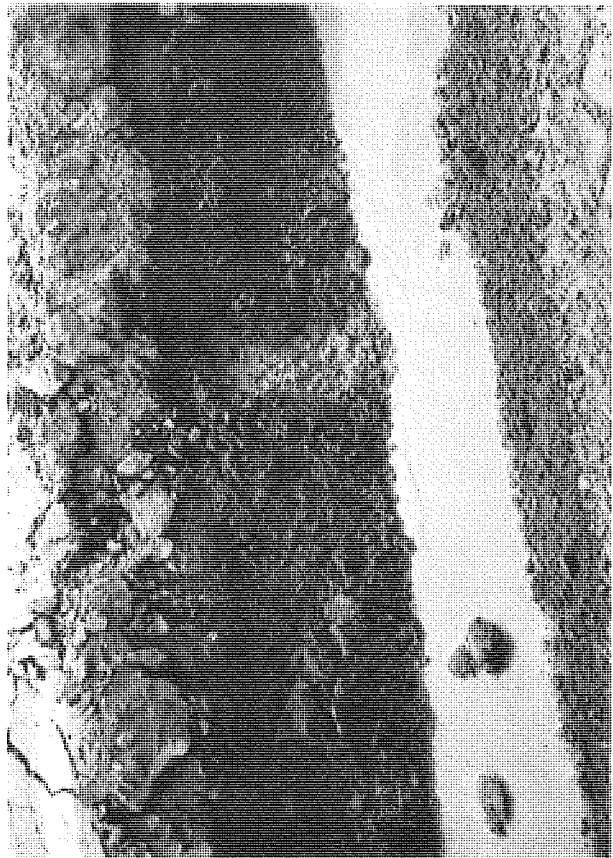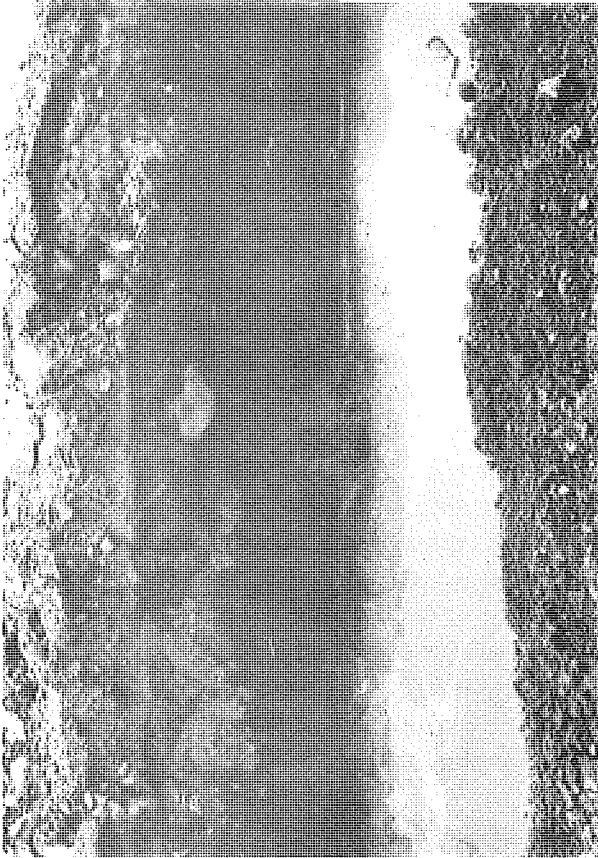
Figure 15. Edge view of pavement on demolition job where shoulder material has been bladed away. The small mud slides caused by water running out of the joints mark the joint's location.

While some 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 15 shows an edge view of an old pavement, taken on a pavement-removal project, after the shoulder had been bladed away. Three adjoining transverse joints were photographed. In the center of the picture in each case is the end of the transverse joint. Note the miniature mud slide that resulted from water running out of the joint.

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 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 many 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 No. 4 rebar at 40 in. centers, with a nominal 1/8 in. saw cut and cold-applied sealant. Bulkhead joints had nominal 1/2 in. diameter hookbolts 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

Figure 17. Widening slab above and shoulder slabs below were placed separately. The stress concentration at shoulder joints still affects cracking in the adjacent longer slab, but naturally to a lesser degree than when integrally cast (I 75 near Birch Run).

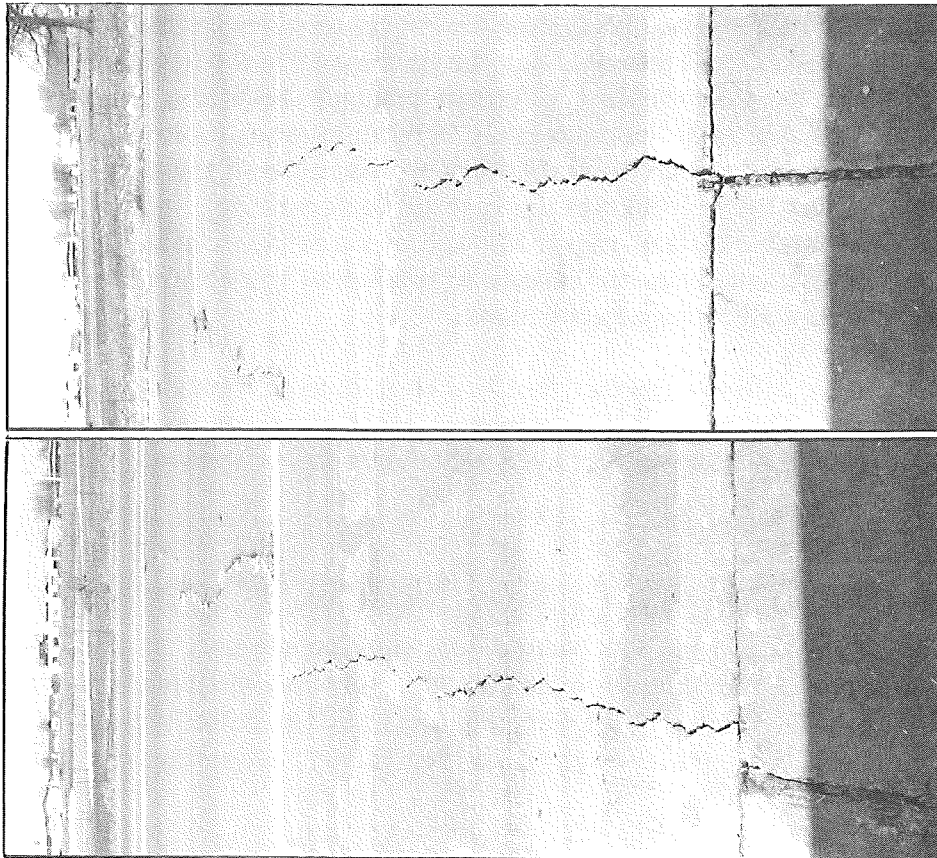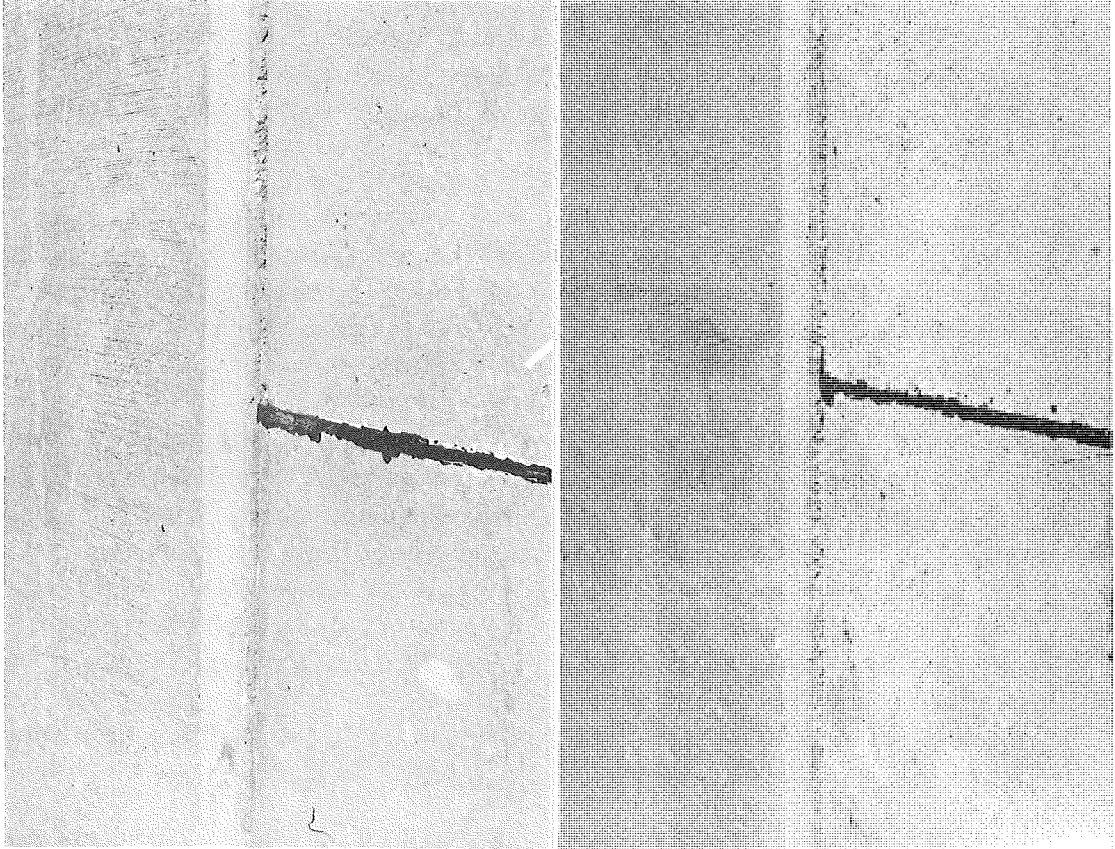Figure 16. The combination of two stress concentrations. I 75 near Birch Run. The two far lanes are old, while the near lane and shoulder were added later in a single pass. Pavement is reinforced with 99-ft slab length and shoulder is unreinforced with slabs about 18 ft long. Shoulder joint plus old slab fracture combine to fracture the newer lane. Obviously, the fractured old pavement has greater effect than the shoulder joint.

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 No. 5 and now are required to be epoxy coated. The saw cut on centerline has been increased to 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 causes additional stress concentrations that generally result in additional or wider cracks. 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. Figure 16 shows a combination of the two factors to break the new pavement slab. In that figure, the pavement widening and shoulder were placed in a single pass. Figure 17 shows an adjoining project where the shoulder was placed separately with a bond breaker between. Cracking still occurs in the main slab, opposite some shoulder joints, but obviously not as severely as with integral paving and old-slab fractures. Michigan procedures now call for a transverse joint in widening slabs adjacent to old slab fractures, and also for bond breakers to be used between pavements and shoulders that are cast separately. Integral casting still is permitted by the Standard Plans but has not been done recently. It is not recommended unless compatible slab lengths are used. Widening an old deteriorated pavement presents numerous conflicting problems in jointing, and also in joint repair at later dates; and it is not intended to lightly treat that subject here.

The effects of the short slabs are more significant when placed adjacent to continuously reinforced concrete pavement (CRCP), since the steel in
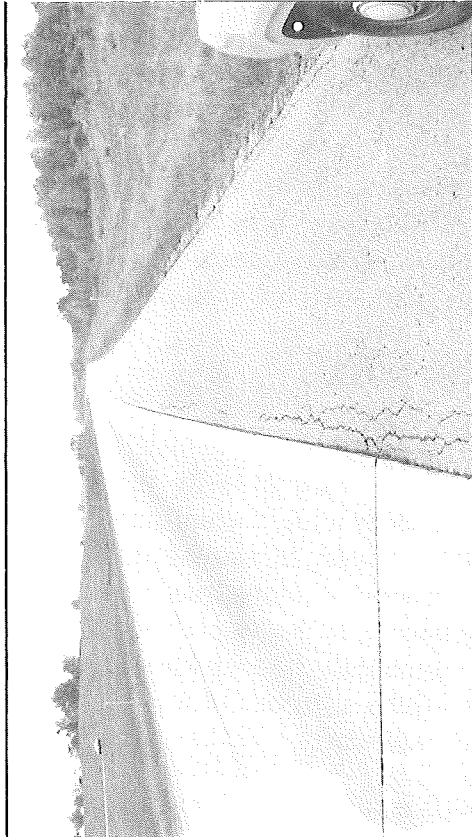
1968 standard bituminous over gravel (170 lb/sq yd)

Full-depth bituminous base course and wearing course
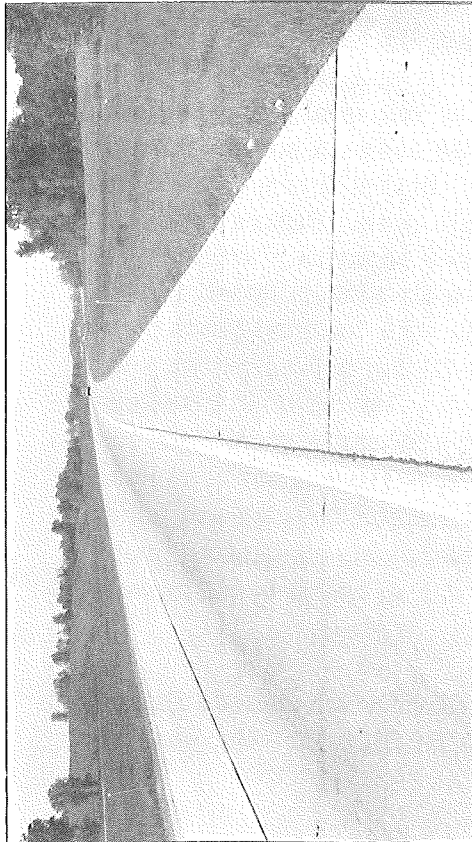
Concrete shoulders
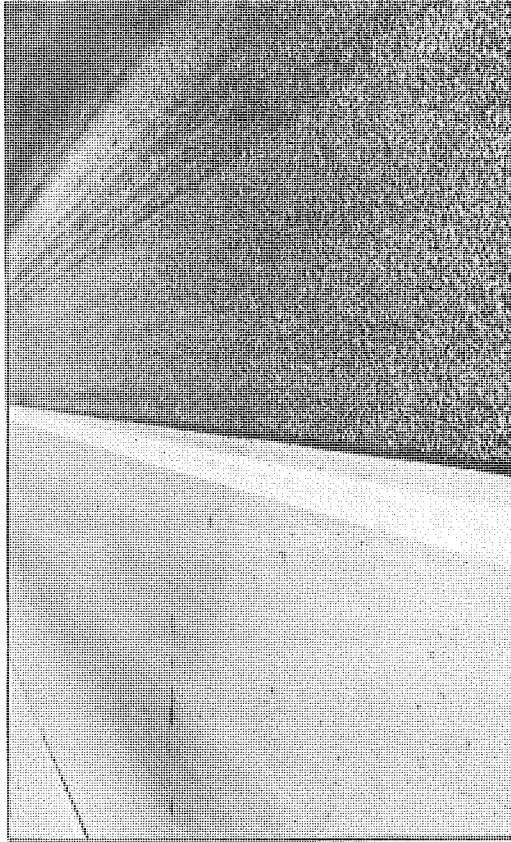
Full-depth bituminous base with seal coat

Figure 18. 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. Commutative cracking of pavement slabs opposite concrete shoulder joints can be seen in many locations on this project, but the cracks still are quite narrow at most locations.

that case is under higher stress and tends to fail early if cracks widen enough to permit salt solution to enter freely. At the present time, we have no concrete shoulder attached to CRCP, and since there is a moratorium on the use of CRCP, it does not appear that any such shoulders will be built in the near future.

The occurrence of commutative cracking in pavement slabs adjacent to short slab shoulder joints is a secondary deleterious effect, and appears to be outweighed by the beneficial effects of having a full-depth structural shoulder and sealed longitudinal joint at the edge of the pavement. The performance of the Illinois I 80 project near Joliet seems to confirm this fact, and although our own experimental section on I 69 still is in quite good condition, the same tendency seems to be developing (15). Figure 18 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 lawsuits 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.

Purely 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 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 up to 21 years (16). 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 subject to damage in handling and installation, and therefore requires extra care, but the performance obtained seems to warrant the required precautions.

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 the 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. Such a program has dramatically reduced the pavement joint blow-up problem in Michigan.

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 encouraged. 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 coarse 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 materials 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 0.1-in. diameter were found to have been moved by such action. Providing drainage seems to be the most reasonable means of combating 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 only secondary deleterious effects, if bond breakers are used between the shoulder and pavement.

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APPENDIX

SELECTION PROCEDURE FOR JOINTS AT REPAIRS,
FOR PREVENTIVE MAINTENANCE PROJECTS
(Pavements With 99-ft Slabs, Base Plates and Poured Seals)

The following process was established to determine the specific locations for joint repairs within each project. The process assumes completion of a joint condition survey record showing structures, ramps, repairs, patches, joint condition category, etc. (see next pages). In order to be noted on the survey as distressed, a joint had to exhibit a spall at least 4 in. wide and 1 ft long. Distressed joints were then categorized by the number of feet of spall plus corner breaks, along the joint.

A set of plan sheets for the project was prepared, showing each joint. All lane joints having 6 or more feet of deterioration were selected for replacement. (This number can be adjusted up or down, depending upon available funds or policy decisions.)

Step No. 1. Record on the plan all existing, full-width pressure relief points, and full-width joints proposed for repair. Such relief points, for example, might be full-width bituminous patches or recent full-width joint or crack repairs where relief has been provided, but will not include expansion joints placed during original construction.

Step No. 2. Record all other proposed repairs (not full-width) of distressed joints or cracks.

Step No. 3. Locate pressure relief joints (PRJ's) or a full-width repair with relief space within 400 ft of structures, if this previously has not been done.

Step No. 4. Locate relief in ramps by replacement of the worst joint in the ramp within 400 ft of the gore. If all of the joints in the 400 ft are in good condition place a PRJ.

Step No. 5. Examine distances between all relief points established or determined in Steps 1 through 4, including those at joint repairs, structures, patches, etc. The spacings between these relief points will then be divided into approximately equivalent distances, not exceeding 850 ft by installing a full-width repair at the worst joint in the vicinity, or by placing PRJ's. Pavement lengths for consideration will begin and end at bridges or similar discontinuities, such as railroad crossings at grade.

The type of joint detail to be used at each side of each repair location (2-in. expansion, 1-in. expansion, or contraction), should be determined as follows:

> Select joint details so that at least 4 in. of expansion space are provided in any 850-ft length of pavement. Use only contraction joints at single-lane repairs, or at other repair locations in multi-lane pavement where expansion space is not provided full-width. (Note that taper sections, acceleration and deceleration lanes, attached concrete shoulders, curb, and gutter are included in the definition of "full-width.") In jobs with numerous full-width repairs, small amounts of expansion space placed close together are preferred to large amounts at isolated locations.

Step No. 6. Field GI of the project (by Design Division and District personnel) for the purpose of making any indicated changes or adjustments.

Step No. 7. Preparation of plans and specifications for the repair projects, showing the location of each repair or PRJ, and the type of joint required at each repair.

DEFINITION OF DIRECTION

LENGTH ALONG JOINT

WIDTHS AWAY FROM JOINT

TRAFFIC FLOW

TRAFFIC FLOW

CORNER BREAKS

SPALL

SPALL

$1''(C_1)$

$4'$ (B_1)

$1'$

$4'$ $1'$

$4''$

$3'$

$2'$

$4''$

$2'$

$1'$

(C_2) $3'$

(B_2) $1'$

LANE NO. 1

LANE NO. 2

LANE 3
(OR SHOULDER)

"LEAVING" EDGE

"APPROACH" EDGE
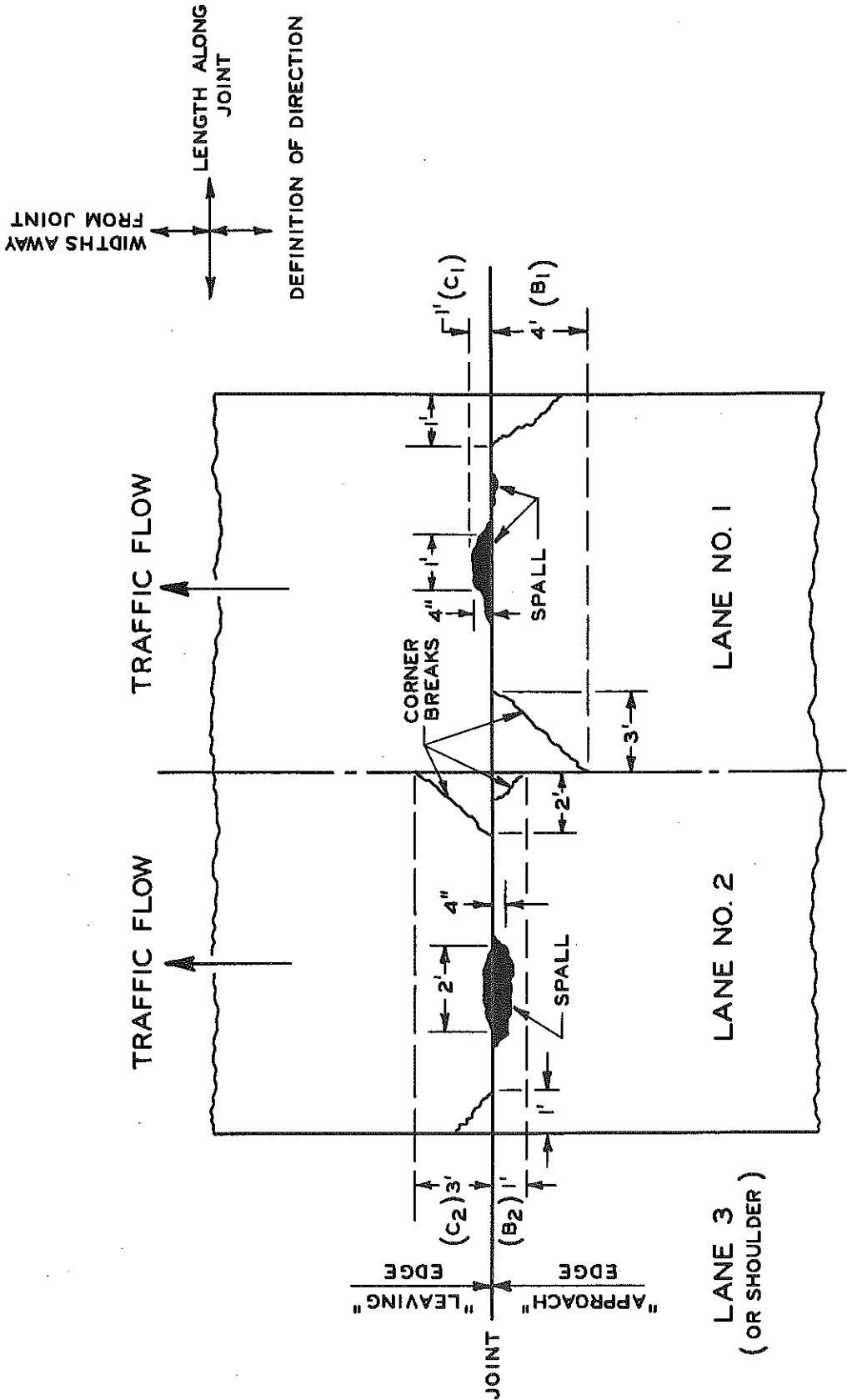
JOINT

RATING – LANE 1, $1 + 1 + 3 = 5$ (A_1)

LANE 2, $2 + 2 + 1 = 5$ (A_2)

Example of deteriorated joint in two-lane one-way road.

- 38 -

PAVEMENT RATING SYSTEM
(MDOT Research Laboratory)

Selection of joints for removal and replacement can be done directly from the survey sheets once the rating is completed.

The system is based on previous experimental work which showed that the probability of blow-up or compressive failure at any joint, is related to the amount and type of observable defects at that joint. Therefore, the system requires that each joint on the project be evaluated, and the rating for each joint recorded. While this may sound complicated, in practice it is quite simple and can be done quickly, and has several distinct advantages.

1) It does not require highly skilled or experienced people to do the rating.

2) Once completed, it gives an objective, numerical value that is a measure of the condition of the project (amount of deterioration per lane mile), and can be used for comparison of projects throughout the State to determine which is in greatest need of repair. Condition ratings can be modified by factors related to traffic densities if desired, for purposes of allocating available funds to the areas of greater use.

3) The rating provides better justification for requests for maintenance funding. This will become more important as FHWA participation in repair contracts increases.

4) The selection of joints for removal can be done from the finished log. Each lane joint that has spall or corner break or black patch extending the full width of the lane is an obvious candidate for replacement. Recent preventive maintenance contracts have selected lane joints with 6 ft or more of deterioration, for replacement.

If funds are limited for a given job, joints can be selected from those with the greater rated deterioration, within the limits of funds available.

5) The finished log of joint ratings can be used directly in the Design Division for preparation of plans and contract documents, which then aid the Project Engineer during the construction phase. Also, the ratings can be updated to reflect repairs made, and used again in the future, to check on the rate of subsequent deterioration of the roadway.

Ratings are made and recorded by lane. For uniformity, Lane No. 1 is the right hand lane in the direction of travel.

Procedures for the rating system and a sample rating form are shown on the attached sheets. Please note that the type of joint used on previous repairs is important for the determination of whether additional pressure relief is required.

SURVEY PROCEDURE INSTRUCTIONS

1) The joint condition survey will be conducted by use of a vehicle equipped with a survey meter. Observations will be made from a vehicle driven on the outside shoulder in the same direction as traffic. Required safety precautions must be followed.

2) Record survey meter reading and point of beginning of project.

3) Record survey meter reading at each patch along with the following information:

    a) Lane in which patch occurs (No. 1, No. 2, No. 3, etc.)
    b) Type of patch (C – concrete, B – bituminous)
    c) Size of patch (longitudinal length x transverse width)
    d) Type of installation (S – sawed joint, NS – not sawed)
    e) Presence of expansion material (E – ethafoam, P – hot or cold-poured bituminous seal over felt filler, C – construction joint no seal, CS – contraction joint with seal).

    EXAMPLE: Two C-4 x 12 – S – E is a concrete patch in passing lane, 4 ft longitudinal by 12 ft transverse, with sawed joints and ethafoam expansion material.

4) Make a tally mark on the survey sheet, ("Good Joints" column), for each joint that has not yet developed spalling or corner breaks of 1 ft along the joint and 4 in. in width from the joint.

5) Record survey meter reading at each joint where the severity of spalling or corner break is 1 ft or more along the joint (accumulated length) and 4 in. or more in width from the joint.

    a) Record accumulated length of spalling or corner break along the joint, in each lane.

    b) Record the width of deterioration from the joint for both sides of joint. (Distance to a saw cut that would remove all deterioration.)

6) Record survey meter reading for all structures, ramp beginnings and endings, (state right or left side), county lines, city limits, etc., and at approximately 1,000-ft intervals on station marks. (This is required so that the same joints can be accurately located if selected for repair.)

7) Make a note if a condition exists where deterioration is not widespread along the joint, but is unusually severe and localized so as to form a hazard if not repaired.

8) Use additional sheets if pavement width is greater than two lanes (ramps, extra lanes, etc.).

9) Record survey meter reading at end of any slab that is mud-jacked or otherwise broken; and, therefore, unfit for placement of a pressure relief joint.

10) Record survey meter reading at any location where a wide crack exists in a slab. ("Wide" here means obviously open, approximately 3/16 in. or more, so that incompressible material can enter and add to pavement "growth.")

JOINT CONDITION SURVEY

Route: _I-96 EB. KENT CO._

Date: _3-31-81_

Personnel: _JOHN DOE_

Footometer Reading	Good Joints	Lane: 1			Lane: 2			Existing Patches Size and Type	Survey Reference Points
		Rating	Width Approach Edge	Width Leaving Edge	Rating	Width Approach Edge	Width Leaving Edge		
000									STA. 100+00
	LHT								
600		5'(A1)	4'(B1)*	1'(C1)*	5'(A2)	1'(B2)	3'(C2)		
	///								
1002		EXAMPLE FROM SKETCH							STA. 110+00
1100								2-C-4X12-SE	
	//								
1320									WEST EDGE, BRIDGE OVER GRAND RIVER
1419									EAST EDGE
	////								
1823									POB ENTRANCE RAMP RIGHT SIDE
	///								
2125									₵ SMITH RD. OVER
	//								
2347									POE ENTRANCE RAMP
2420								1-B-3X12-NS	
	////								
2932									KENT-IONIA CO. LINE

* NOTE: a) MINIMUM PATCH IS 6' BY LANE WIDTH, IF B+C EXCEEDS 6' THEN REQUIRED PATCH LENGTH IS B+C.

b) IF PATCHES IN ADJACENT LANES DON'T LINE UP, MAKE AT LEAST 2-FT OFFSET, OTHERWISE MAKE THEM LINE UP SO THAT SAW-CUT RUN-OUT IN OTHER LANE HAS LESS PROBABILITY OF CAUSING A LOSS OF THE CORNER OF THE REMAINING SLAB.

c) DO NOT PLACE PRJ IN SLABS THAT HAVE BEEN MUD JACKED, OR ARE BADLY FRACTURED. PRJ's SHOULD BE A LEAST 10 FT FROM TRANSVERSE CRACKS OR JOINTS, IF POSSIBLE.