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CURRENT DESIGN OF CONCRETE PAVEMENT IN NEW JERSEY

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SYNOPSIS

This paper pertains to the design of concrete pavement currently employed in New Jersey in the construction of highways that are to carry heavy truck traffic. The design features described and discussed are: (1) pavement thickness; (2) slab length and width; (3) reinforcing steel; (4) dowelling of expansion joints; (5) dowel coatings; (6) joint filler; and (7) sub-base construction. Information is given in regard to the performance of certain expansion joints employed in the past. Discussed are: (1) the effects of corrosion on dowels; and (2) the reasons why the design includes expansion joints. Included are: (1) traffic data; (2) the results of load-transfer tests on various sizes, shapes and lengths of dowels; and (3) the general behavior of contraction joint pavement.

The development of a design of concrete pavement that, on a long-term basis, will resist the damaging effects of heavy truck traffic has for a number of years been a problem of major importance in New Jersey. Because of New Jersey's particular geographical location on the Atlantic Seaboard, and in the most highly industrialized area in the country, its primary highways carry an unusually large amount of traffic. A case in point is U. S. Route 1 which, in the vicinity of Newark, carries an average of 56,000 vehicles daily. Of greater significance, however, is the fact that on a number of these primary highways approximately 25 percent of the traffic consists of truck traffic. Furthermore, approximately 10 percent of the total traffic consists of tractor semi-trailer combinations which, in New Jersey, may have a legal gross weight of 60,000 lb. At this legal gross weight, these 3-axle combinations have been found to have an average load of 26,000 lb., on each rear axle. However, axle loads much in excess of 26,000 lb., are common. In conjunction with 13½-in. dual tires an axle load of 34,400 lb., is currently legal.

Traffic volumes and the general make-up of the traffic at various locations on U.S. Route 1 are shown in Table 1. Traffic volumes and axle loadings on several primary highways are shown in Table 2. It should be noted, however, that the data shown in Table 2 do not include locations where traffic is heaviest, because conditions in these locations do not permit loadometer surveys to be made.

In recent years, as observed elsewhere throughout the country in general, both the number and the average weights of trucks have increased materially, and are still on the increase. As indicated in the foregoing, New Jersey's design problem is one that not only concerns high axle loads but also one that concerns a very high frequency of high axle loads.

EFFECT OF TRUCK TRAFFIC

As amply demonstrated on a country-wide basis in recent years, the predominating tendency of heavy truck traffic in passing over a

TABLE 1
TRAFFIC DATA—U. S. ROUTE 1
1947

	N. J. Route 25 Newark	N. J. Route 25 Linden	N. J. Route 25 Metuchen	N. J. Route 26 Penns Neck
Total Traffic (All vehicles), Approximate Daily Average	55,600	48,000	20,000	11,000
Passenger Vehicles	42,940	37,310	14,350	8,160
2 Axle Single Unit, Less than 1½ tons	1,410	2,290	735	380
2 Axle Single Unit, 1½ tons and over, Single Rear Tires	280	10	12	115
2 Axle Single Unit, 1½ tons and over, Dual Rear Tires.	5,690	4,610	1,540	1,015
3 Axle Single Unit	316	410	317	85
Tractor and Semi-trailer Combinations, 3 axles	4,790	3,170	2,850	1,205
Tractor and Semi-trailer Combinations, 4 axles	160	200	186	30
Tractor and Semi-trailer Combinations, 5 Axles	7	0	5	8
Full Trailers .	7		5	2

1931. Our investigations indicate that, in general, in conjunction with older pavements having inadequate dowelling of the joints, and on subgrades susceptible to pumping, a daily truck traffic exceeding aproximately 150 axle loads of 8 tons or more will cause damage to the pavement.

Although faulting and pumping have usually occurred concurrently, the experience in New Jersey has been that, under heavy truck traffic, pumping is not essential to the occurrence of faulting. Pumping is, however, a process that materially contributes to faulting inasmuch as it results in a progressive removal of subgrade material from under the ends of slabs and thereby makes possible the advancement of faulting to a pronounced degree. The experience has also been that, under very heavy truck traffic, if the ends of slabs at concrete pavement is to cause faulting at joints and cracks and, in addition, to cause pumping of the subgrade material from under the pavement at these points. These forms of failure first occurred extensively in New Jersey in joints and cracks are not securely connected together faulting will occur in conjunction with almost any natural subgrade material, irrespective of whether the subgrade is susceptible to pumping or not. Among other

TABLE 2
TRUCK TRAFFIC DATA—VARIOUS ROUTES
(1947)

	U. S. Route 1 N. J. Route 25 Metuchen	U. S. Route 130 N. J. Route 25 Cranbury	U. S. Route 130 N. J. Route 25 Pennsauken	U. S. Route 46 N. J. Route 6 Great Notch	U. S. Route 22 N. J. Route 28 Clinton	U. S. Route 22 N. J. Route 29 No. Plainfield	N. J. Route 17 Ramsey
Total Traffic (All vehicles)—Approximate Daily Average	20,000	7000	15,000	12,000	9300	22,000	9250
Total Trucks—Approximate Daily Average	5650	2720	4130	1340	2000	3590	1900
Approximate number 5 tons and over	5000	2530	3040	870	1690	2970	1190
" " 10 " " "	3510	2010	2020	580	1370	1910	700
" " 15 " " "	2250	1500	1170	380	1080	1560	460
" " 20 " " "	1410	1060	870	230	830	1180	330
" " 25 " " "	620	450	290	70	360	360	270
" " 30 " " "	340	160	80	30	150	70	150
" " 35 " " "	140	50	20	10	50	0	20
" " 40 " " "	110	20	10	0	10		0
" " 45 " " "	30	10	0		0		
" " 50 " " "	0	0					
Total Axle Loads—Approximate Daily Average	13,450	6750	9220	2000	4450	7380	4650
Approximate number 2 tons and over	12,500	6350	7320	1500	3960	6480	3480
" " 4 " " "	6980	3710	3910	760	2360	3560	1800
" " 6 " " "	4450	2770	2440	460	1870	2440	1080
" " 8 " " "	3080	2110	1800	300	1470	2030	810
" " 10 " " "	1450	1250	1000	180	840	1060	660
" " 12 " " "	640	470	260	70	240	350	380
" " 14 " " "	90	120	50	30	80	130	160
" " 16 " " "	50	30	10	10	10	0	30
" " 18 " " "	30	10	0	0	0		10
" " 20 " " "	0	0					0

things, it is primarily these forms of failure that the design of pavement described and discussed in this paper is intended to prevent.

PROPERTIES OF THE CONCRETE

Inasmuch as the quality of the concrete which can be produced in any given locality can greatly influence the choice of design, it appears desirable to comment on the typical concrete produced in New Jersey.

Almost all of the failures that have occurred in connection with concrete pavements in New Jersey have been of a structural nature, as differentiated from failures attributable to weakness or disintegration of the concrete itself. This is not to say that the concrete is everywhere perfect.

There are, for example, a few pavements that have undergone excessive expansion (growth) which, in some instances, have proved very troublesome. But the concrete in conjunction with these "growers" is, nevertheless, still very strong and free from disintegration. A few pavements have been damaged by the attack of de-icing agents. However, these instances are in the minority; the over-all picture being sound, durable concrete. The typical concrete has a 28-day compressive strength exceeding 5000 psi. This high quality of the concrete is due to a plentiful supply of first-class materials, plus careful control in their selection, testing and use. The coefficient of thermal expansion is approximately 0.000005 per degree, F.

A noteworthy example of the high strength and durability of the typical concrete produced in New Jersey is the oldest section of concrete pavement on the highway system. This section, 0.90 miles in length, a portion of which is shown in Figure 1, was constructed in 1912. This pavement was constructed wholly as an experiment, in a location where a macadam road had repeatedly failed during spring thaws. Actually it is not much more than a crude covering of unreinforced concrete ranging from 4 inches to 8 inches in thickness constructed directly on the failed macadam road. As will be noted, after 36 years of service it is still functioning as an unresurfaced concrete pavement. Although badly cracked in some locations, the concrete itself is still as sound as ever. Test cores have shown that the present compressive strength averages 8300 psi.

Primarily because of the high strength and durability of the concrete at our disposal we have adopted a long-range viewpoint in regard to design—the objective being to find ways and means to effectively counteract, on a long-range basis, all of the forces and conditions that have been found to be damaging to this inherently sound material.

Figure 1. 1912 Pavement

PAVEMENT THICKNESS

All pavements that are to carry very heavy truck traffic now are constructed 10 in. thick. Under traffic conditions less severe, and on more stable subgrades, the pavements are constructed 9 in. thick. In line with standard practice in New Jersey since 1921, the cross section in all cases is uniform. These thicknesses have not, however, been determined on the basis of calculation. On the contrary, because of the many very complex factors that are involved, the effects of which are extremely difficult to evaluate, we feel that we have not as yet accumulated sufficient fundamental data to calculate the required thickness. It is believed that many more experimental pavements need to be constructed, and that much additional research is necessary, before any dependable method of calculation can be developed. For these reasons we are still obliged to turn to the various pavements in service and to use them as a guide in the selection of thickness. However, the 9- and 10-in. thicknesses currently specified have been employed in New Jersey for a number of years and, on that basis, appear to be adequate.

REINFORCING STEEL

Reinforcing steel is included in the design primarily to prevent the detrimental opening

of cracks. Past experience has shown that open cracks are very damaging. Inasmuch as open cracks and undowelled expansion joints are essentially in the same category, the pavement adjacent to an open crack will, in the presence of heavy trucking, inevitably undergo faulting. And an open crack is, of course, capable of admitting large amounts of surface water to the subgrade and susceptible to becoming partly or totally filled with incompressible earthy materials such as silt and sand, both conditions being obviously undesirable.

In the current design the longitudinal members consist of ⅜-in. deformed bars spaced 7½ in. c. to c., or, as an alternative, an equivalent cross-sectional area of cold-drawn wire in the form of welded wire fabric. Installation is made 2 in. below the pavement surface. This amount and design of reinforcing steel has been standard for some years.

SUB-BASE

Our experience indicates that, within reasonable limits, the thickness of the pavement is of much lesser importance than the stability of the subgrade. We have, for example, 10-in. pavements on poor subgrades that have undergone serious failure and, on the other hand, 9-in. pavements on stable subgrades that, under comparable conditions of traffic, have performed very well. We have found, furthermore, that most of the subgrade soils in New Jersey undergo some differential expansion when frozen and that they tend to become unstable during periods of thaw. For these reasons, all heavy-duty pavements are currently constructed on a layer of granular sub-base material having a minimum thickness of 12 in.

On native soils of low bearing value the sub-base is supplemented with an additional underlying layer of selected material which, as a general rule, is also essentially granular in composition. The sub-base is constructed in two courses. First, a bottom course, 7 in. thick, in which the elutriable content is limited to a maximum of 5 percent. Second, a top course, 5 in. thick, in which the elutriable content is limited to 7 percent. It is specified that a minimum of 45 percent of the material in the top course shall be retained on the No. 4 sieve. This top course is, in a sense, an armor coat. The design has the following purposes:

1. To provide a layer of sub-base material immediately under the pavement that contains a sufficient amount of large, stable granular material as to not be susceptible to pumping.
2. To provide a sub-base that may be rendered highly stable by compaction equipment.
3. To provide a sub-base that will reduce differential frost-heaving to a minimum and which will in itself be highly resistant to the unstabilizing effects of freezing and thawing.
4. To provide a sub-base that, by reason of substantial thickness, will prevent, or at least effectively minimize, freezing of the underlying native soil.

In conjuction with a joint highway research project with Rutgers University and the Public Roads Administration, equipment is being developed and fabricated at the University to study the effects of vibration on various kinds of subgrade and sub-base materials.

SLAB LENGTH

Under normal conditions, the current standard length of slab is approximately 80 ft. Dowelled expansion joints having wood filler are installed in conjunction with these slabs. The design does not include intermediate contraction joints. Some of the factors that influenced the adoption of this length are:

1. The results of studies indicating that when pavements are constructed on granular sub-base materials, an 80-ft. length of slab is not excessive.
2. The construction of pavements on granular sub-base material, which has been standard practice since 1939, has been found to have effected a very marked reduction in cracking. The effect of these sub-base materials has been to minimize differential frost-heaving which, with older pavements constructed directly on silty, clayey soils, has been a major cause of cracking.
3. The observed capacity of the standard amount of reinforcing steel to maintain cracks in older pavements at hair-line width.
4. The results of a number of experimental pavements constructed during 1946 and 1947 which included slabs of various lengths, and which have thus far indicated that a slab length of 80 ft. is not excessive. These pavements were constructed in various parts of the state, and the slabs range in length, in incre-

ments of 11 ft., from 56 to 133 ft. The purpose of these experiments is to determine how much the length of slab may be increased without increasing the standard amount of reinforcing steel.

During construction, gauge plugs were installed at many of the joints in conjunction with all lengths of slabs and considerable data have been accumulated in connection with daily and seasonal changes in slab length. These data indicate that, in New Jersey, concrete pavement slabs normally undergo an over-all yearly change in length of approximately 0.045 in. per 10 ft. of length.

A number of tests have been made to determine the sliding resistance of pavement slabs on various kinds of subgrade and sub-base materials. The results indicate that sand offers the least resistance and that the coefficient in this case ranges from approximately 1.3 to 1.5. A coefficient as high as 3.0 has been found in conjunction with bank-run gravel, slightly damp when placed and compacted to a high degree of density. In this instance the high coefficient was found to be due primarily to the bonding of the concrete to large particles of gravel at the subgrade surface—the bottom of the slab being, as a result, very rough and jagged. For these reasons the current specifications pertaining to sub-base construction require that a thin layer of concrete sand be spread over all areas where large stones, gravel, or other large aggregates are exposed.

In New Jersey, the installation of intermediate contraction or warping joints to control cracking does not appear to be warranted.

SLAB WIDTH

The current normal width of slab is 12 ft. This corresponds with the present width of lane provided for traffic. Widths of 13 and 14½ ft. in which no longitudinal cracking has occurred have been employed in past years.

DOWELS

The dowels currently used in expansion joints consist of 1¼ in. diameter cold-finished steel bars, approximately 18 in. long. These are installed 12 in. c. to c. To prevent their corrosion, the dowels are encased in corrosion-resistant metal tubing throughout both the unbonded part of their length and the part that passes through the joint space. The general features of the dowels, and their positioning in the concrete, are shown in Figure 2. The specifications permit the use of either stainless steel or Monel metal tubing, but the joint manufacturers have thus far furnished Monel. The tubing is of 20 gauge thickness. Prior to attachment, the tubing is slightly under-size. By means of a press, the tubing is forced on the bar, the resulting fit being so close and tight that the bar and the tubing are, in effect, a single unit. At the time of installation, to effect free movement in the concrete, the tubing is given a light brush coating of asphaltic oil, grade MC-2. A large variety of possible dowel lubricants were tested, such as paints, oils and greases. From the standpoint of effectiveness, ease of application and adhesion to the dowels, this material has proved the most satisfactory.

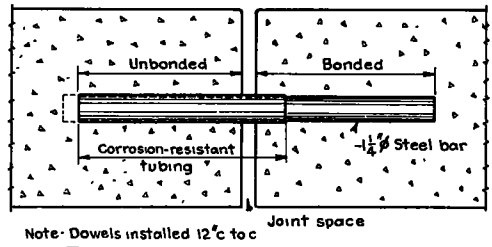

Figure 2. Current Design of Dowel

The principal factors that determined the employment of this design of dowel are as follows:

1. Under conditions of heavy truck traffic, ¾-in. round dowels have failed to prevent serious faulting, even where installed on 10-in. centers. It has been found that in some locations where pumping has been severe, and despite a 10-in. spacing, many of these dowels have actually broken at the joint space. Breakage is believed to be due primarily to repeated reversals of stress exceeding the elastic limit. However, in many instances corrosion of the dowels at the joint space no doubt played an important part in the breakage. Because of extensive serious faulting, the installation of ¾-in. round dowels was abandoned in New Jersey in 1934.

2. The progressive development of extremely high resistance to the movement of dowels consisting of 2-in. structural channels. This is in reference to a design of joint installed extensively during the period 1934—42, the

principal features of which are shown in Figure 3. The fact that these channel dowels were susceptible to progressively developing ex-

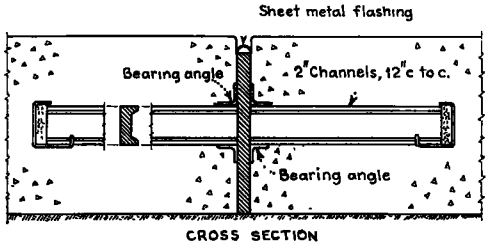

Figure 3. Channel-Dowel Joint

tremely high resistance to movement did not become apparent until the fall of 1944, at which time excessively wide faulted cracks were discovered in a section of pavement 6½ years old in which these joints had been installed. At these cracks the longitudinal reinforcing steel had failed. Following this discovery, gauge plugs were installed at all joints and transverse cracks in a ½-mi. section of this pavement. Subsequent measurements disclosed that the joints underwent almost no opening during pavement contraction and that, instead, contraction was being taken care of by opening of the cracks. Later, the concrete adjacent to some of the "frozen" joints was removed and, by means of a special dowel-pulling device, it was determined that in some cases the sliding resistance per dowel exceeded 20,000 lb.

This extremely high resistance to movement had come about despite the fact that prior to installation the dowels had been given two coats of paint, white lead followed by red lead, plus a supplemental coating of transmission oil at the time of installation. The progressive development of extremely high resistance to movement has also occurred in conjunction with 2-in. channel dowels coated with tar paint. Subsequent investigations and tests have confirmed the fact that this seizure of the dowels has resulted from corrosion (rusting), the mechanics of which will be discussed later.

3. The unsatisfactory performance of concrete sills constructed under the pavement at joints, more particularly: During the last war, due to the steel shortage, and for purposes of experiment, several miles of pavement were constructed in which dowelling of the expansion joints was omitted. Instead, concrete sills, 5 ft. wide and approximately 10 ft. long, were constructed under the slab ends. The design of sill employed on one heavy-trucking route is shown in Figure 4. Elsewhere the sills were of the same dimensions in plan, but of unreinforced concrete uniformly 4 in. thick. In some cases the tops of the sills were constructed flush with the bottom of the pavement. These sills have not proved satisfactory, for the following reasons:

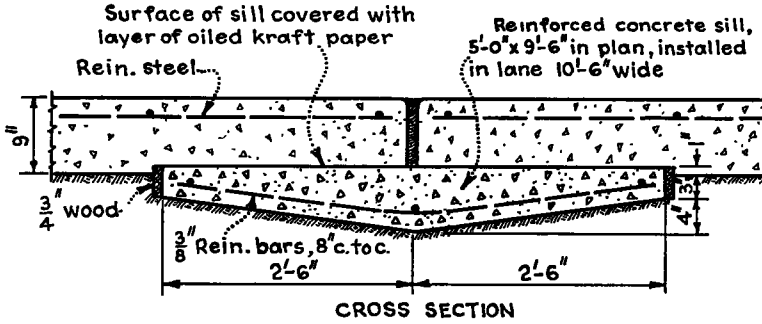

Figure 4. Sill Joint

(a) During the winter, even in conjunction with granular sub-base material, frost-action results in excessive differential vertical displacement of the slab ends (faulting). It has been found that at many of these joints the differential displacement does not disappear when the frost leaves the subgrade. Apparently this is due to the infiltration of solid materials between the sills and the pavement when in the heaved condition.

(b) Under heavy truck traffic, the sills have not prevented a detrimental amount of fault-

ing; faulting of as much as ⅜-in. having occurred after three years of service (sill type shown in Figure 4). Under these conditions the sills are apparently driven down into the subgrade.

(c) In some cases, transverse cracking of the pavement has occurred directly over the edges of the sills.

It is, therefore, primarily because of (a) the inadequate strength of ¾-in. round dowels, (b) the corrosion and seizure of 2-in. channel dowels, (c) the unsatisfactory performance of sill joints, and (d) the need to employ dowels of substantial dimensions, that the current design of dowel was adopted. The diameter and length were selected on the basis of tests made to determine the relative load-transferring capacities of various structural shapes. The typical test setup is shown in Figure 5. These tests, the results of which are shown in Table 3, indicate that a 1¼-in. diameter dowel is practically the optimum size of round dowel for use in 9-in. pavement. On the basis of these tests, a 1¼-in. round dowel acting across a joint space of 1½-in. (the greatest width of joint space anticipated) has, within the proportional limit, a load-transferring capacity of at least 8000 lb. Installation is made on 12-in. centers primarily as a factor of safety. At this spacing, it is probable that at least three dowels come into full play under the heavier dual-tired wheel loads. Considering the hundreds of thousands, and even millions, of reversals of stress to which the dowels in a heavy-trucking pavement are subjected within the course of a few years, plus the fact that if a joint device fails there is no practical way of replacing it, a high factor of safety appears warranted.

The possibilities of other designs of corrosion-resistant dowels are, however, being investigated. On an experimental basis, there are in service several joints in which 1½-in. stainless steel I-beams were installed. These I-beams are not, however, commercially available at present, but may be in the near future. The use of a structural shape of stainless steel appears very promising. The possibilities of tubular dowels consisting of seamless, carbon-steel tubing partly encased in corrosion-resistant metal are also being investigated.

Other experimental installations include joints in which 1¼-in. diameter hot-rolled dowels were installed. These dowels were treated in various ways to render them corrosion-resistant. To prevent them from in-

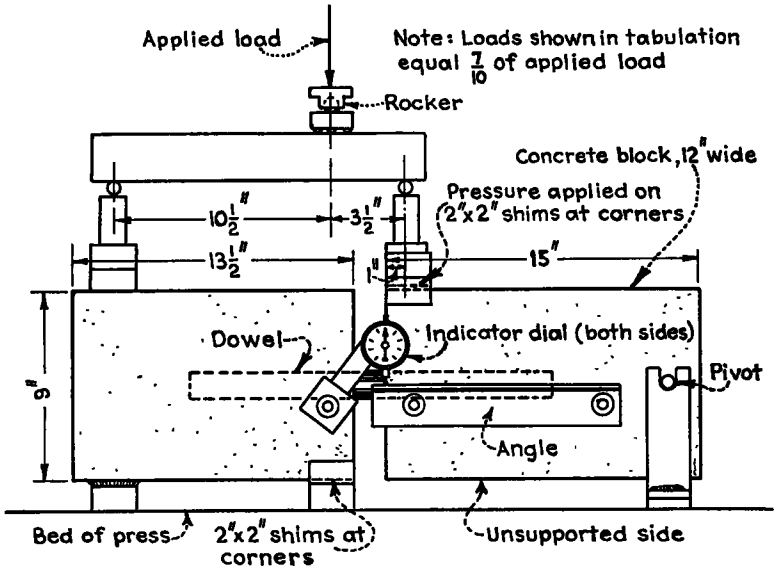

Figure 5. Dowel Test Setup

TABLE 3
DOWEL TESTS

Dowel Data		Set-up Data						Test Data															
Shape, Size, Surface Finish	Length	Joint Opening	1- by 1- by $\frac{1}{4}$-in Bearing Angles	Embedment in Conc. Each Side	Area	Moment of Inertia	Section Modulus	Concrete Strength	Deflections in inches for Various Loads in pounds										First Spall	First Crack	Maximum Load	Deflection at Max. Load	Proportional Limit[a]
									2000	4000	6000	8000	10,000	12,000	14,000	16,000	18,000	20,000					
	in.	*in.*		*in.*	*sq. in.*			*psi.*											*lb.*	*lb*	*lb.*	*in.*	*lb.*
$\frac{3}{4}$ in. Round—hot rolled	20	$\frac{7}{8}$		$9\frac{3}{16}$	.442	.016	.041	4300	.009	.020									4,950	None	5,500	.250	4,200
$\frac{1}{2}$ in. " —hot rolled	$21\frac{1}{2}$	$1\frac{1}{2}$		10	.442	.016	.041	4940	.019	.050									4,100	None	5,300	600	3,200
1 in. " —hot rolled	20	$1\frac{1}{2}$		$9\frac{1}{4}$	.785	.049	.098	5060	.010	.020	.036	.250							5,600	8,300	8,400	500	5,300
1 in. " —cold finished	18	$1\frac{1}{2}$		$8\frac{1}{4}$	.785	.049	.098	5500	.004	.012	.023	.042	.170						6,300	6,300	10,500	260	6,500
$1\frac{1}{4}$ in. " —cold finished	$21\frac{1}{2}$	$1\frac{1}{2}$		10	1.227	120	.192	4400	.006	.012	.019	.027	.035	.048	.080				7,700	12,200	14,400	.115	11,500
$1\frac{1}{4}$ in. " —cold finished, SAF-C1045	18	$1\frac{1}{2}$		$8\frac{1}{4}$	1.227	.120	.192	4860	.008	.016	.023	.032	.045	.060					8,500	12,700	13,900	.095	9,500
$1\frac{1}{4}$ in. " —hot rolled galvanized	$17\frac{3}{4}$	$1\frac{1}{2}$		$8\frac{1}{8}$	1.227	.120	.192	5150	004	.009	.015	.023	.041	.104					9,100	9,100	12,600	.200	8,000
$1\frac{1}{4}$ in. " —cold finished, Monel clad	$17\frac{3}{4}$	$1\frac{1}{2}$		$8\frac{1}{8}$	1.227	.120	.192	5210	.005	.011	.017	.024	.036	.053	.096				7,700	7,000	14,700	.180	8,500
$1\frac{3}{8}$ in. O.D. cold-drawn tubing 1 in I.D.	18	$1\frac{1}{2}$		$8\frac{1}{4}$	.700	.127	.184	3750	.006	.013	.024	.039	.059	.101					7,700	9,800	13,300	.230	9,500
$2\frac{3}{8}$ in. O.D. " " " $2\frac{1}{16}$ in. I.D.	$21\frac{1}{2}$	$1\frac{1}{2}$		10	1.089	.674	.567	4420	.006	.011	.016	.020	.025	.030	.038	.075	.150		9,800	12,200	17,500	.155	14,000
1 in. Square—hot rolled	$17\frac{1}{2}$	$1\frac{1}{2}$		8	1 000	.083	.167	4940	.009	.016	.024	.032	042						8,400	11,000	12,100	.100	8,400
1 in. " —stainless steel	$17\frac{1}{2}$	$1\frac{1}{2}$		8	1.000	.083	.167	5230	.009	.016	.026	.040							8,300	9,500	9,900	.135	5,000
$1\frac{1}{2}$ in. by $\frac{1}{2}$ in —hot rolled	$17\frac{1}{2}$	$1\frac{1}{2}$		8	.750	.141	.188	4730	.008	.016	.025	.036	.065						7,000	10,000	10,000	.070	7,000
$1\frac{1}{2}$ in. by $\frac{1}{2}$ in.— " "	$17\frac{1}{2}$	$1\frac{1}{2}$	4–12 in long	8	.750	.141	.188	5140	.006	.013	.019	.026	.036	.115					None	12,900	13,175	.165	9,800
$1\frac{1}{2}$ in. by $\frac{5}{8}$ in — " "	$17\frac{1}{2}$	$1\frac{1}{2}$		8	.938	.176	.235	5770	007	013	.019	.026	.049						8,500	9,900	10,200		8,400
$1\frac{1}{2}$ in. by $\frac{5}{8}$ in — " "	$17\frac{1}{2}$	$1\frac{1}{2}$	4–12 in. long	8	.938	.176	.235	4830	.004	009	.014	.019	.027	.040	165				None	14,200	14,400	.250	9,800
$1\frac{1}{2}$ in. by $\frac{5}{8}$ in — " "	$17\frac{1}{2}$	$1\frac{1}{2}$	4–3 in. long	8	.938	.176	.235	5000	005	.010	.016	022	.028	040					12,500	11,600	13,100	140	9,800
2 in. by $\frac{5}{8}$ in.— " "	$17\frac{1}{2}$	$1\frac{1}{2}$		8	.750	.250	.250	4850	.008	016	.023	.032	.044	.160					8,200	9,000	12,200	.180	9,200
2 in. by $\frac{5}{8}$ in — " "	$13\frac{1}{2}$	$1\frac{1}{2}$		6	.750	.250	.250	5070	.010	.017	.025	.034	.045						11,700	9,900	11,800	.150	9,200
2 in. by $\frac{5}{8}$ in — " "	$9\frac{1}{2}$	$1\frac{1}{2}$		4	.750	.250	.250	4710	.010	.021	.032	.052							7,000	7,400	8,500	.060	7,000
2 in. by $\frac{5}{8}$ in.— " "	$17\frac{1}{2}$	$1\frac{1}{2}$		8	1.250	.417	.417	4970	.004	010	.015	.019	.023	.028					13,200	10,300	13,700	.040	11,500
2 in. by $\frac{5}{8}$ in.— " "	$13\frac{1}{2}$	$1\frac{1}{2}$		6	1.250	.417	.417	5450	.004	.008	.012	.017	.021	.034					None	9,200	12,200	.038	11,500
2 in. by $\frac{5}{8}$ in.— " "	$9\frac{1}{2}$	$1\frac{1}{2}$		4	1.250	.417	.417	4760	.006	.014	.020	.026	.040						10,000	9,200	10,400	.060	8,800
2 in. by 2 in. Tee—hot rolled	$17\frac{1}{2}$	$1\frac{1}{2}$		8	1.047	.370	.261	4400	.003	.005	008	.012	.022	.060	120				11,300	13,500	15,800	.250	8,700
$1\frac{1}{2}$ in. I-beam, $\frac{7}{8}$ in. flange, $\frac{1}{4}$ in. web—stainless steel	18	$1\frac{1}{2}$		$8\frac{1}{4}$	.563	.172	.229	5730	.004	.008	.012	.017	.022	.030	.054				12,950	9,800	15,890	200	12,000
$1\frac{1}{2}$ in. I-beam, $\frac{7}{8}$ in. " $\frac{1}{4}$ in. "— " "	15	$1\frac{1}{2}$		$6\frac{3}{4}$	.563	.172	.229	5820	.004	.008	.012	.017	.024	.034	.057	.162			13,300	9,100	16,100	.170	12,000
$1\frac{1}{2}$ in. I-beam, $\frac{7}{8}$ in. " $\frac{1}{4}$ in " — " "	12	$1\frac{1}{2}$		$5\frac{1}{4}$	.563	.172	.229	6160	.004	.009	.014	.020	.029	043	.126				11,200	9,100	14,700	.300	11,000
2 in Channel, $\frac{5}{8}$ in flange, $\frac{1}{4}$ in. "—hot rolled	20	$\frac{7}{8}$	4–12 in. long	$9\frac{1}{4}$	.641	.282	.282	4580	.003	.006	.010	.013	017	.020	.024	.029	.036	046	None	13,500	22,700	070	15,700
2 in " $\frac{7}{8}$ in. " $\frac{1}{2}$ in. " — " "	20	$2\frac{1}{2}$		$9\frac{1}{4}$	1.234	.501	.501	4290	.004	.007	.010	.013	.016	.019	.023	.028			None	15,400	16,700	034	14,700
2 in. " $\frac{7}{8}$ in. " $\frac{1}{2}$ in. " — " "	20	$1\frac{1}{8}$	4–12 in. long	$9\frac{1}{4}$	1.234	.501	.501	3970	.002	.004	.007	.010	.013	.016	.018	.022	.028	.034	None	13,700	24,800	.064	15,400

[a] The loads shown in the last column indicate the maximum loads at which the deflections were essentially directly proportional to the applied loads.

fluencing each other, every other joint in these series has dowels encased in Monel metal. By means of periodic measurements taken at gauge plugs installed at the joints, those dowels, if any, that subsequently develop resistance to movement will be identified.

LONGITUDINAL JOINTS

The design of longitudinal joint consists of a tongue-and-groove keyway. In view of the high stability of the sub-base materials now employed, tie bars are not installed. However, the question of whether or not tie bars are needed is being studied.

JOINT FILLER

Wood joint filler is currently specified, for the following reasons:

1. Laboratory tests indicate that wood filler has a high degree of recovery after compression.
2. The resistance that wood filler offers to the expansion of the pavement is considered beneficial.
3. Because of capacity to recover, the wood filler is expected to counteract the damaging effects of whatever solid, foreign materials infiltrate into the joint spaces.
4. The success of wood filler reported over a period of years in various parts of the country.

Current specifications permit the installation of any kind of clear sound lumber, with the proviso that all lumber that is not heartwood cypress, redwood, or western red cedar shall be given preservative treatment.

DOWEL CORROSION

As mentioned previously in connection with channel-dowel joints, serious difficulties have been experienced with these joints as a result of rusting. The fact that rusting of the embedded, unbonded portion of a dowel can greatly restrain its normal movement in the concrete has been found to be due to the following circumstances:

1. The volume of the rust that forms on an ordinary steel dowel is several times the volume of the metal destroyed.
2. When confined, rust is capable of exerting a tremendous expansive effort. In the case of a dowel, this expansive effort results in a counter action in which the concrete, in effect, grips the dowel and restrains its movement.

The mechanics of this phenomenon are indicated in Figure 6 which shows a block of concrete six years old that has been split apart by the rusting of embedded steel bars. Prior to pouring the concrete around the bars that pass through this block they were given a liberal coating of grease and for several months thereafter could be slid back and forth by hand. But, being stored out-of-doors, moisture gained entrance to the embedded parts of the bars and rusting occurred. Because of the expansive effort of the rust, and the resulting high pressures, the bars soon became practically immovable. As shown, the block finally has been split apart in several locations by this action.

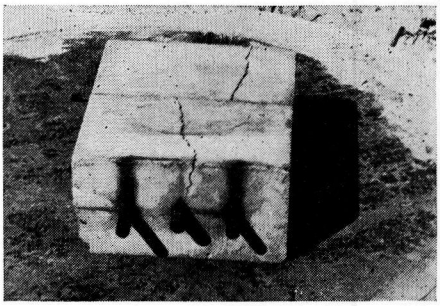

Figure 6. Splitting of Concrete Block Due to Rusting of Embedded Bars

In the case of the channel-dowel joints, it is very likely that the rusting of the bearing angles above and below the dowels contributed to the restraint to dowel movement inasmuch as the angles presented additional surfaces susceptible to rusting. In addition, the metal-to-metal contact between the angles and the dowels may have increased the rate of corrosion. Studies in this connection indicate that joint devices in which the dowels or load-transferring members are in association with bearing angles or the like are more susceptible to rusting and seizure than the conventional dowel joint. The familiar rusting fast of bolts and nuts is an example of what tends to occur when unprotected metal parts are in association with one another. Manifestly, the greater the amount of metal that is included in the dowels or any related parts, and the higher the load-transferring efficiency of the device, the greater the need to prevent rusting.

Regarding dowel corrosion in general, everyone appears to appreciate the fact that unpro-

tected steel undergoes corrosion when exposed to the elements. On the other hand, very little thought seems to have been given to the question of what happens to dowels that, of necessity, are obliged to function year after year under conditions very favorable to corrosion, expecially that part of the dowel which passes through the joint space. In addition to normal dampness, the salt-laden water that leaks into joint spaces as a result of the use of de-icing agents probably tends further to promote corrosion. Aside from seizure, it appears to have been generally overlooked that ordinary steel dowels are highly susceptible to undergoing a partial or total loss in load-transferring ability within a matter of a few years simply as a result of corrosion having effected a reduction in their cross-sectional area. In New Jersey, ¾-in. round dowels, after from 12 to 14 years of service, have been found to be in all stages of corrosion—the amount of corrosion ranging from that which appears to be unimportant to that at which the metal at the joint space has been almost totally destroyed.

The phenomenon of corrosion-fatigue, in which a small amount of corrosion brings about the premature failure of metals subjected to repeated reversals of stress, may also warrant investigation in connection with dowels.

In regard to restraint to dowel movement, it is believed to be evident that any such restraint is reflected in a corresponding amount of increase in the tensile stresses in the pavement and in the reinforcing steel passing through cracks, and that, furthermore, if the pavement contains only enough reinforcing steel to overcome subgrade friction any material amount of restraint will result in over-stressing and failure of the reinforcing steel. This is not to say that wherever failures of the reinforcing steel have occurred that failure has necessarily been due to dowel restraint, there being many other possible causes. But excessive restraint, whether it be the result of misalignment, improper lubrication, or rusting of the dowels, is a condition that obviously should be avoided. Actually the question of how to effect the continued free movement of large dowels, at the least cost, has been for several years New Jersey's most difficult joint design problem.

DOWEL LUBRICANTS

In view of the importance of unrestrained dowel movement, some remarks pertaining to New Jersey's experiences and tests in connection with dowel lubricants appear warranted.

It is generally recognized that to effect the free movement of dowels it is necessary that they be either coated with a lubricant or encased in sleeves of some kind, for two reasons, namely; to prevent the adhesion or bonding of the concrete to the dowel metal; and to neutralize the effect of mechanical imperfections in the surfaces of the dowels, such as are common to hot-rolled bars. Experiences and tests in New Jersey indicate the following:

1. Light oils are not satisfactory lubricants for use in conjunction with hot-rolled dowels because, even though they may prevent bonding of the concrete to the dowels, they lack sufficient body or thickness to neutralize mechanical imperfections—considering that well-consolidated concrete flows into intimate contact with the dowel surfaces and therefore conforms with any imperfections present. The placement and consolidation of the concrete around dowels tends to remove the lubricant and, for this reason, light oils cannot be depended on to prevent bonding effectively.

2. Heavy oils will effectively prevent bonding and usually neutralize imperfections, but they are difficult to apply uniformly and usually run down the sides of the dowels and collect in excessive amounts underneath the dowels.

3. Greases are difficult to apply uniformly and, in excessive amounts, both oils and greases weaken the concrete immediately around the dowels. Apparently because of their soap content, some greases undergo a material loss in lubricating qualities when in association with concrete.

4. A coating of pigment paint, such as red lead, that is allowed to dry, does not promote free movement. In fact a dried coating of any paint is an unsatisfactory lubricant. In addition, tests have shown that because of (a) mechanical imperfections, (b) the intimate contact of the surrounding concrete, and (c) the back and forth movement of the dowels, pigment paints eventually become reduced to a paste or powder affording little or no protection against rusting. As indicated previously, the most serious instances in New

Jersey of dowel rusting and seizure have occurred in connection with dowels that were given two coats of pigment paint—white lead followed, when dry, by red lead.

5. The fact that a coat of paint prevents the concrete from coming into direct contact with the dowel metal is no guarantee whatever that it will effect free movement—that is, if the coating is dry. This was demonstrated very convincingly in New Jersey in 1934 in connection with channel dowels that were given a brush coating of tar paint several weeks prior to installation. This material had been specified in view of its apparent success in the past in conjunction with ¾-in. round dowels. Being completely dry at the time of installation, these coatings proved almost totally ineffective. Because of the resulting excessive restraint to the opening of the joints, very serious, extensive transverse cracking occurred within a matter of days. Pull-out tests made in the field disclosed that, despite this coating, some of the dowels offered as much as 30,000 lb. resistance to movement. Subsequent laboratory tests disclosed that concrete will bond to a dowel that has a thin, hard, dry coating of paint almost as effectively as to an uncoated dowel. These tests furthermore disclosed that if, for example, a pull of 5000 lb. is required to cause a dowel to move, and if it is moved, say ⅛in. by this pull, further movement will not usually occur at a pull materially less. That is, the mere breaking of the bond, so to speak, does not necessarily effect a substantial reduction in resistance to movement.

As indicated in the foregoing, dowel coatings should possess lubricating qualities. They should not, however, be so thick as to result in excessive clearance, nor be of such composition as to weaken the concrete around the dowels. Although there may be no entirely satisfactory dowel lubricant, in New Jersey's experience asphaltic oil, grade MC-2, applied at the time of installation, has thus far been found the best. The proper viscosity of this material is maintained by adding kerosene, when necessary.

EXPANSION JOINTS

Because of the current rather widespread practice of omitting expansion joints and substituting contraction joints in their place, it appears appropriate to explain why this has not been done in conjunction with the design of pavement described in this paper.

Offhand it may appear that the question at stake is whether or not concrete pavements require expansion joints. This, however, is not the case at all. On the contrary, the question really is: Do joints installed at relatively long intervals, such as the 80-ft. interval employed in the design described, require a filler? In terms of New Jersey's experience, they definitely do, for the following reasons:

It has been found that, on a long-term basis, it is impossible to prevent the infiltration of incompressible foreign materials such as silt and sand into joint spaces. These materials are apparently carried into the joint spaces primarily by the action of water. As a general rule these materials do not accumulate uniformly throughout the joint spaces and, consequently, they offer localized or eccentric restraint to the closure of the joints. For example, the part of the joint space that usually first becomes filled with foreign material is the part adjacent to the shoulders.

Inasmuch as the joints in association with 80-ft. slabs constructed during the summer undergo an opening, in New Jersey, of approximately ⅜in. during the winter these joints are obviously very susceptible to infiltration. And if, as in the case of contraction joints, the joint spaces do not contain a compressible filler to counteract the excessive localized restraint that infiltrated materials offer to the closure of the joints during warm weather, the pavement is very likely to undergo spalling, longitudinal splitting, buckling or total rupturing at the joints; the latter condition being the well-known "blow-up". These forms of damage are quite common in older pavements in New Jersey in which, after some years, the joint filler has disappeared. It has been found that infiltration is most likely to cause damage in pavements that have narrow joint spaces in association with long slabs, as would be the case if contraction joints were substituted for expansion joints in the design of pavement described.

Another factor that bears on this question is the matter of growth. The fact that every concrete pavement is at least potentially capable of growth raises the question of whether or not some allowance for this contingency

should be included in the design. Although it may be true that expansion joints will not permanently protect a pavement that undergoes growth they nevertheless will delay its damaging effects. The experience in New Jersey has been that, in general, blow-ups are the result of the combined effects of growth and infiltration at joints. Consequently it appears that the characteristics of the joint filler can play a very important part in determining whether or not blow-ups will occur, even in the event of growth.

Expansion joints are, therefore, included in the current design primarily to protect the pavement from the damaging effects of infiltration and to provide some measure of relief in the event of growth. They also have the added function of providing spaces that, in the event of excessive infiltration, are wide enough to be cleaned out. The fillers are not installed to permit the pavement to have complete freedom to expand—if this were the case, wood filler would not have been specified. Considering that wood filler will not undergo appreciable compression until subjected to a pressure of from 300 to 600 psi., these joints are not expansion joints in the true sense of the word. The beneficial effects of restraint to the expansion of the pavement are fully appreciated. But the practical problem of effecting restraint and, at the same time, maintaining it within safe limits remains, in our opinion, still to be solved.

CONTRACTION JOINTS

The construction of pavements having contraction joints has never been standard practice in New Jersey. However, during the past six years, primarily to determine the general behavior and possibilities of this design, a number of miles of pavement having undowelled, dummy-groove contraction joints have been constructed. Some of these pavements have no expansion joints whatever whereas the others have expansion joints at intervals ranging from 105 to 343 ft. The contraction joint interval ranges from 15 to 17 ft.

Inasmuch as these pavements were constructed in locations where truck traffic is relatively light, and because of their limited period of service, their ability to carry heavy trucking is not yet known. However, during construction, gauge plugs were installed at a great many of the joints and considerable precise data have since been obtained in regard to the amount of joint opening and seasonal changes in joint width.

The principal reasons why contraction joint pavements are not normally constructed in New Jersey are:

1. Construction in recent years has been primarily for heavy trucking.

2. Under heavy trucking, it appears very doubtful that aggregate interlock can in itself be depended on to prevent excessive faulting, even if the contraction joints are spaced as little as 15 ft. apart and expansion joints are omitted. Since, under these traffic conditions, it appears necessary to install joint devices in conjunction with contraction joints, and since these devices would have to be of essentially the same design as installed at expansion joints, a close spacing of the joints would necessarily result in a substantial increase in the cost of joint devices per square yard of pavement. A close spacing would also increase the cost of joint maintenance, detract from the appearance and riding qualities of the pavement and introduce a greater number of points of potential failure.

3. The installation of contraction joints at relatively long intervals would result in considerable opening of the joints during cold weather and, in consequence, total loss of aggregate interlock during these periods. Therefore, in conjunction with long joint intervals the installation of joint devices would be an absolute necessity, and the resulting longer slabs would necessitate the installation of reinforcing steel. However, the principal objection to this design is the greater amount of daily and seasonal changes in joint width and the consequent greater difficulty of maintaining the joints sealed. These factors materially increase the possibility of serious damage to the pavement as a result of the infiltration of incompressible foreign materials. Actually, considering the difficulty, and perhaps impossibility, of preventing infiltration, it appears probable that any contraction joint pavement, regardless of the joint spacing, is at least potentially susceptible to damage by infiltration.

Some of the more important observations concerning the behavior of contraction joint pavements constructed in New Jersey are as follows:

1. Cracking below the grooves does not occur simultaneously at all of the joints. Consequently there is a period during early life when some of the joints undergo opening and some do not.

2. If the weather conditions are such that the concrete can attain considerable strength before an appreciable lowering in pavement temperature occurs, cracking at and opening of some of the contraction joints may be long delayed. For example, a 420-ft. section of 8-in. pavement having contraction joints at 15-ft. intervals, but no expansion joints, was constructed during Oct. 1948. The grooves were 2½-in. deep. The "as constructed" temperature of the pavement was approximately 60 F. During the following two weeks the air temperature at no time fell below 40 F., and the mean daily air temperature was at no time lower than 48 F. By Dec. 3 cracking and opening had occurred at only one-third of the joints. The pavement temperature at this time was 41 F., and the average opening of the joints that had opened was .080 in., the maximum being .125 in.

3. Even in later life all of the joints do not open the same amount. For example, the joint openings measured in Jan. 1948 (pavement temperature, 32 F.) at 18 consecutive contraction joints spaced 17 ft. apart in the central portion of a 3000-ft. section of pavement having no expansion joints, constructed in May 1946, were as follows: .071, .086, .066, .108, .104, .093, .076, .082, .093, .083, .070, .058, .095, .085, .046, .073, .128, and .058 in. In June 1948 (pavement temperature, 80 F.) the corresponding joint openings were: .023, .037, .023, .054, .035, .027, .030, .017, .016, .023, .033, .014, .045, .036, .017, .034, .088 and .026 in. Note that the second joint from the last was open .128 in. in Jan. 1948 and that it was open .088 in. the following June. Considering the amounts that these joints are open during the winter, it appears doubtful that effective aggregate interlock exists during these periods, especially at the second joint from the last.

4. As found elsewhere throughout the country, where expansion joints are installed in conjunction with contraction joints, the expansion joints soon undergo considerable permanent closure which is reflected, more or less proportionately, in increased opening of the intermediate contraction joints. For example, in a pavement constructed in May 1946, having expansion joints with 1-in. cork filler at 340-ft. intervals and intermediate contraction joints at 17-ft. intervals, the expansion joints were closed an average of .42 in. two months later, July 19 (slab temperature, 97 F.). In Jan. 1948 (slab temperature, 32 F.) 20 contraction joints (10 joints on each side of a 1-in. cork joint) were open an average of .113 in., the openings ranging from .070 to .164 in., the cork joint being closed .293 in. Assuming that the .293 in. of expansion joint closure is reflected in a proportionate increase (.014 in. per joint) in the contraction joint opening it can perhaps be assumed that had the expansion joints been omitted the average joint opening would have been .099 in. Effective aggregate interlock at this joint opening could hardly be expected.

The question of whether or not the omission of expansion joints will result in the pavement being under compression depends primarily on the following factors:

1. The "as constructed" temperature of the pavement.
2. The shrinkage of the concrete.
3. The permanent shortening of the slabs as a result of prolonged subjection to high compressive stresses during hot weather (plastic flow).
4. The extent to which the joints are restrained from closing.
5. The amount of growth.

In New Jersey, as elsewhere, most pavements are constructed during the warmer seasons and, consequently, they generally have a high "as constructed" temperature. Due to the combined effects of high air temperature, sunlight, and the heat generated within the concrete itself during the setting and hardening process, most pavements constructed during the summer have an "as constructed" temperature of not less than 80 F., and very frequently in excess of 100 F.

Inasmuch as a pavement tends to exceed its "as constructed" length only during those times when its temperature exceeds its "as constructed" temperature, it follows that a contraction joint pavement constructed during the summer cannot be under compression during the cooler seasons, provided, of course, that the joints are free from infiltration or other forms of restraint to their closure and that the pavement has not undergone growth.

Furthermore, inasmuch as pavements constructed during the early spring and late fall very rarely have an "as constructed" temperature of less than 50 F., it follows that even these pavements, barring restrained joints and growth, cannot be under compression during the winter.

Tending further to diminish the development of compressive stresses within the pavement is the factor of shrinkage which necessarily tends to shorten the slabs and create expansion space. This factor appears to vary considerably with the materials, amount of mixing water, and the climate. However, in New Jersey, the amount of shrinkage is, at the most, very small. In addition, perhaps because of a fairly uniform rainfall throughout the year, no very significant changes in slab length occur as a result of seasonal changes in moisture content.

Tests have shown that, as a result of plastic flow, concrete subjected to a high stress over a prolonged period tends to undergo permanent deformation. The effect of the stress is, however, much more pronounced during the very early life of the concrete. For this reason, the slabs in a contraction joint pavement constructed during the spring, in cool weather, are much more likely to undergo a permanent shortening during the following summer than in the case of pavements constructed at other times. It is evident that this phenomenon also contributes to the creation of expansion space and to the reduction of compressive stresses. Tests indicate, however, that as concrete attains age it becomes almost totally resistant to plastic flow. Consequently, on a long-term basis, plastic flow apparently cannot be depended on to relieve excessive compressive stresses.

In New Jersey, the materials that infiltrate into joint spaces usually consist of relatively clean silt and (or) sand having high internal friction which, in thin layers, are practically incompressible. In the event of infiltration, contraction joint pavements in New Jersey would, therefore, be under considerable compression during hot weather. However, inasmuch as infiltration is a slow process (if the joints are kept sealed) our contraction joint pavements apparently will not begin to enjoy the benefits of compression until some indefinite time in the future—compression being a desirable condition that some appear to believe is brought about automatically by the simple omission of expansion joints. Apparently, from the standpoint of promoting compression, the sealing of contraction joints has an adverse effect.

As mentioned previously, some pavements in New Jersey have undergone growth. These pavements are, however, much in the minority. Contraction joint pavements could, therefore, be constructed without the risk of growth causing extensive blow-ups, at least during early life. (There are, however, some indications that over a period of many years most concrete pavements tend to undergo a certain amount of growth.) Therefore, in general, and at least for a number of years, growth could not be expected to induce compression in the pavement. Consequently, with few exceptions, compression during the cooler seasons could come about only as a result of infiltration during cold weather, which is a very uncertain and risky process at best. If, however, the pavement could be induced to undergo rapid growth during very early life, in an amount just sufficient to compensate for the opening of the joints, a very desirable condition might obtain.

Based on hundreds of measurements taken of the seasonal opening and closing of joints, there are as yet no indications that any of the contraction joint pavements in New Jersey are under compression during the cooler seasons.

In view of the foregoing, it is believed to be evident that it would be impractical in New Jersey to construct slabs of such a length as would insure adequate aggregate interlock under conditions of heavy truck traffic. Even in the case of a pavement having no expansion joints, contraction joints spaced as closely as 15 ft. apart would, in general, be open an average of approximately 0.08 in. during cold weather. Furthermore, as already indicated, the contraction joints in any given section of pavement vary considerably in the amounts that they open. Consequently, in a series of contraction joints that are open an average of 0.08 in., some of the joints would be open considerably more than 0.08 in. The effectiveness of the aggregate interlock at these more open joints would clearly be just about nil. (It is, in fact, very seriously doubted that aggregate interlock will prevent excessive faulting if the contraction joints open 0.10 in.

or more, and probably even less.) Therefore, even in a series of closely spaced contraction joints, it appears necessary to install load-transfer devices at all of the joints, if only to avoid the faulting of those that open excessive amounts. However, in order to settle this question, the construction of a ¾-mi. experimental section of 10-in. unreinforced pavement having undowelled contraction joints at 15-ft. intervals, and no expansion joints, is now under contract. This section is in a location where it will carry considerable heavy trucking, and will be constructed on 12 in. of granular sub-base placed over 6 in. of selected material.

Reference was previously made to appreciable delay in the opening of dummy-groove contraction joints under stable temperature conditions. Tests indicate that if cracking below the groove is long delayed the configuration of the fracture may be such as to materially reduce the effectiveness of aggregate interlock—for the reason that a crack that occurs during early life tends to follow a circuitous path around the larger aggregate whereas if the concrete has attained considerable tensile strength the crack tends to pass through the aggregate, the latter resulting in a more nearly plane surface. To induce early cracking, a ¾-in. strip of wood, approximately 1¾-in. high, was installed on the subgrade, immediately below the top groove, at most of the contraction joints installed in New Jersey. This no doubt had some effect, but it did not eliminate erratic cracking and opening.

GENERAL CONCLUSIONS

1. In terms of New Jersey's experiences, a design of concrete pavement that, on a long-term basis, is expected to resist the destructive effects of heavy truck traffic, and other so-called "natural" destructive forces, should, in addition to adequate thickness, include the following supplementary features:

(a) A very stable subgrade composed of materials that are resistant to both pumping and the unstabilizing effects of freezing and thawing.

(b) Joint devices of adequate strength to prevent faulting and which are resistant to corrosion.

(c) Reinforcing steel in an amount sufficient to prevent the detrimental opening of cracks.

(d) Joint fillers of such composition as will tend most to counteract the damaging effects of infiltration.

2. The corrosion of dowels results in a loss in their load-transferring ability and, under certain conditions, the development of excessive resistance to their movement—the latter being a very important factor in connection with large dowels.

3. Appreciable restraint to dowel movement, whether as a result of misalignment, inadequate lubrication, or corrosion, tends to result in over-stressing and failure of the reinforcing steel passing through cracks.

4. Dowel coatings should possess lubricating properties. A dried coat of any kind of paint is a very ineffective dowel lubricant.

5. Subgrade friction ("subgrade drag") is the least on sand subgrades.

6. In New Jersey, in order to prevent damage due to the infiltration of solid materials, joints spaced at relatively long intervals require a filler.

7. In New Jersey, even if expansion joints are omitted, contraction joints spaced as little as 15 ft. apart undergo opening during cold weather to the extent that effective aggregate interlock is highly doubtful.

8. In any given series of contraction joints, the amount of joint opening is very erratic, some of the joints opening much more than others.

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