An Experiment in Pavement Slab Design

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A number of experimental features were introduced in a portland cement concrete pavement on a regularly scheduled construction project by the Illinois Division of Highways in 1952. Included in the experimental portion of the project were sections of nonreinforced pavement with and without transverse joints, and reinforced sections having the Illinois distributed steel reinforcement with a variety of transverse joints and joint treatments. Although the pavement is still in excellent condition, certain early behavior trends are now indicated.

●THE EXPERIMENTATION with which this report is concerned involves the introduction and subsequent study of the behavior of several experimental features in a portland cement concrete pavement constructed on a regularly scheduled paving contract in 1952.

A major motivating factor behind the early planning of this research study was the uncertainty at that time as to the continued availability of steel reinforcement for concrete pavement because of the demands of warfare in Korea. Distributed steel reinforcement had been used in Illinois pavement since the late 1930's, except for a period during World War II when it was not available. Pavement placed during the war-time years without steel and with expansion joints at close intervals had not proved satisfactory in Illinois experience. The experimentation was therefore directed in a large measure toward a study of design types suitable for development as substitutes for the standard reinforced pavement. Other items of research include several types of transverse contraction joints under study to determine service behavior with respect to riding quality, durability and retention of seal; and two types of joint-sealing compound being evaluated qualitatively.

The construction project selected for the study is 4.3 mi long and located on US 66 immediately south of Springfield. It is identified officially as Section 110X-5, SBI Route 126, Federal Project FI-166(18), Sangamon County. US 66 is one of the heaviest traveled rural pavements in the state and at the location of the test project carries presently about 12,600 vehicles per day, including 2,200 trucks and buses. The test pavement consists of two 12-ft lanes carrying southbound traffic. It is separated from the northbound pavement by a grassed median 30 ft wide.

The topography of the area of the test pavement is generally flat and the pavement lies on a slight embankment throughout most of its length. Soils are fine grained and reasonably uniform, classifying mostly as A-6 and in the higher plasticity range of the A-4 group.

The major construction operations on the test project included the placing of the embankment, a 6-in. trenched granular subbase, and the 24-ft width of experimental pavement. No special effort was made to obtain particular uniformity and precision of construction other than to observe the usual construction and inspection techniques of the Illinois Division of Highways.

The test pavement is a part of a Federal-participating construction project and the experimentation has been conducted in cooperation with the U. S. Department of Commerce, Bureau of Public Roads.

RESEARCH PURPOSES

The following research purposes were established for the study:

1. To make observations and comparisons of the behavior of the 1951 standard Illinois design of concrete pavement having a 10-in. thickness, welded-wire fabric reinforcement, and hand-edged full-depth metal-plate contraction joints at 100-ft intervals versus certain types of plain (nonreinforced) concrete pavement. The following sections of plain pavement were included for this purpose:
   a. A 10-in. uniform thickness without joints;
   b. A 9-in. uniform thickness without joints;
   c. A 10-in. uniform thickness with dummy-groove plane-of-weakness contraction joints at 20-ft intervals (no load-transfer devices); and
   d. A 9-in. uniform thickness with dummy-groove plane-of-weakness contraction joints at 20-ft intervals (no load-transfer devices).
2. To make comparative observations of the behavior of the Illinois standard full-depth metal-plate contraction joint and plane-of-weakness joints of the following basic types:
   a. Dummy-groove (with load-transfer devices); and
   b. Sawed (with load-transfer devices).
3. To make comparative studies of the riding quality and durability of full-depth metal-plate contraction joints having hand-tool finishing of joint edges, versus the same type of joint without hand-tool finishing.
4. To make comparisons of the effectiveness of a hot-poured rubber-asphalt joint sealer versus a cold-applied rubber-asphalt ready-mixed sealing compound.
5. To evaluate the effect of adhesion between sealing compound and concrete that might result from abrading opposing vertical faces of edged contraction joints.
6. To evaluate the comparative behavior of wetted-burlap cure and impermeable paper cure in the control of cracking prior to sawing transverse joints.

A statewide change in construction practice with respect to transverse contraction joints while the experimental construction work was in progress resulted in the control section being divided into two parts. One part was constructed with hand edging used at the transverse metal-plate joints as was the practice at the time the project was initiated. Hand edging was omitted on the other portion in accordance with the new practice that went into effect during construction.

To accomplish these research purposes, 11 test sections inclusive of the two control sections varying in length from about 1,000 ft to about 3,600 ft were established. They were constructed consecutively, the pavement being placed in a five-week period during May and June 1952.

PROJECT DESIGN

The general layout of the construction project is shown in Figure 1. Test sections, as previously mentioned, are built to serve as the two southbound lanes of the dual highway. Out-to-out width of these two lanes is 24 ft. They are on new location lying west of and parallel to the formerly existing 20-ft portland cement concrete pavement built in 1932 to serve traffic in both directions. Under a 1952 construction contract the old pavement was widened with a 4-ft strip of plain portland cement concrete placed on the west side, and the full 24-ft width was surfaced with 3 in. of I-11 bituminous concrete. The new surfacing serves northbound traffic.

A typical cross-section of the portion of the roadway occupied by the experimental pavement is shown in Figure 2. Shoulders are of turf, 6 ft wide between pavements and 10 ft wide outside of the pavements. Frontage roads are provided along each side of the throughway.

LOCATION AND LAYOUT OF TEST SECTIONS

It has been indicated previously that the experimental research features are included as design components of the southbound lanes of a dual highway. Principal

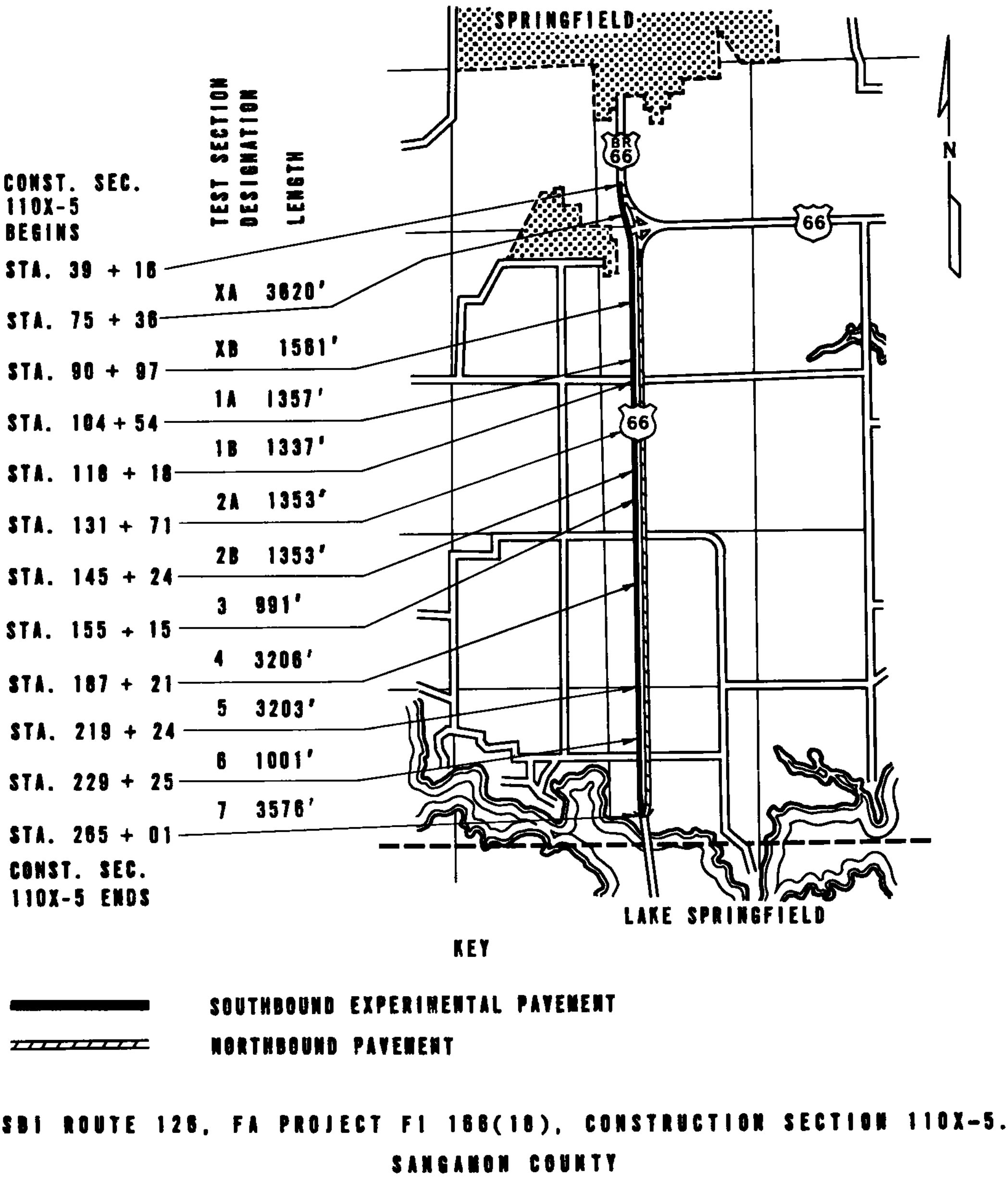


Figure 1. Location and layout of experimental sections.

design features and other pertinent data regarding the test pavements are given in Table 1.

Several features of design are common to all test sections. Some of these common features are apparent in Table 1 but others are not. Those not otherwise shown are as follows:

1. All pavement is uniform 24-ft width portland cement concrete.
2. A trench-type, undrained, granular subbase underlies all test sections. The

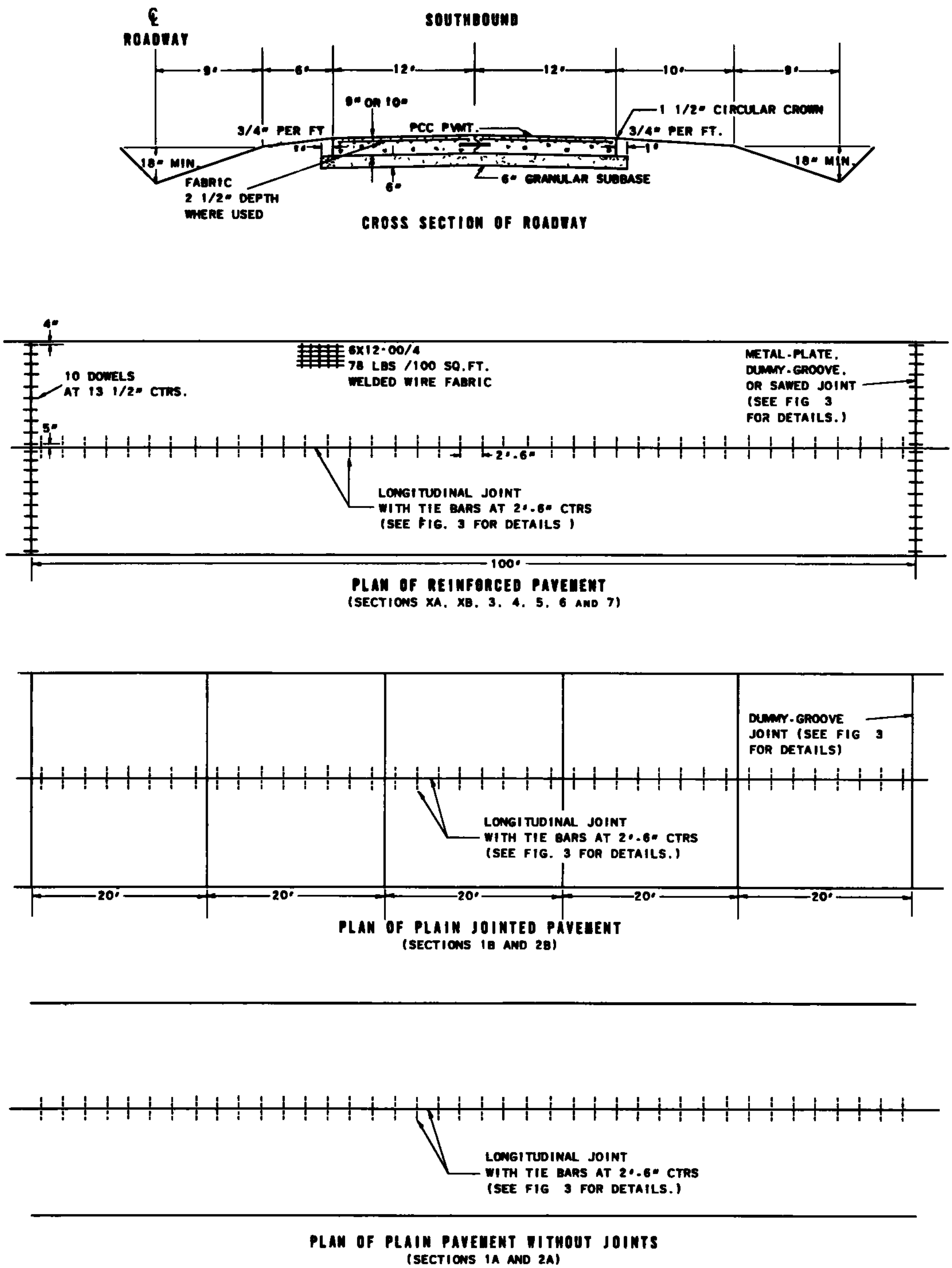


Figure 2. Typical cross-section of roadway, and plan sections of portland cement concrete pavement.

TABLE 1

PRINCIPAL DESIGN FEATURES OF TEST SECTIONS

Test Section	Station to Station	Length (ft)	Pavement Thickness (in.)	Reinforcement (lbs per 100-sq ft)	Panel Length (ft)	Joint Type	Load Transfer Devices
XA	39+16 to 75+36	3620	10	78	100	Metal plate (unedged)	Dowels
XB	75+36 to 90+97	1561	10	78	100	Metal plate (edged)	Dowels
1A	90+97 to 104+54	1357	10	None	-	Construction only	None
1B	104+54 to 118+18	1337	10	None	20	Dummy-groove	None
2A	118+18 to 131+71	1353	9	None	-	Construction only	None
2B	131+71 to 145+24	1353	9	None	20	Dummy-groove	None
3	145+24 to 155+15	991	10	78	100	Sawed	Dowels
4	155+15 to 187+21	3206	10	78	100	Dummy-groove (abraded faces)	Dowels
5	187+21 to 219+24	3203	10	78	100	Dummy-groove	Dowels
6	219+24 to 229+25	1001	10	78	100	Sawed	Dowels
7	229+25 to 265+01	3576	10	78	100	Metal plate (edged; abraded faces)	Dowels

Note: All transverse joints sealed with cold-applied rubber-asphalt compound except 18 joints of Section 7 sealed with hot-poured rubber-asphalt compound granular subbase has a design thickness of 6 in., and is 26 ft wide, extending 1 ft beyond the edges of the pavement on each side.

3. Longitudinal joint assemblies are of the metal-plate type with No. 5 deformed tie bars 2 ft 6 in. long at 2-ft 6-in. centers.

The design features in standard use in Illinois during the period of planning this experimental research are represented throughout several of the test sections so that the behavior of each experimental feature of the study that is a departure from the then-current Illinois standard may be compared, not only with that of other experimental types, but also with the behavior of the respective standard. Essential features of Illinois portland cement concrete pavement design in use at that time (1951) were previously discussed. None of the experimental features of this research except possibly the spacing of sawed contraction joints at 100-ft intervals, were innovations. However, none had been used previously in Illinois under the heavy traffic conditions anticipated for this pavement.

Experimentation Included in the Test Sections

Following are brief discussions of the study items that are departures from 1951 Illinois standard practice and which are the principal components of the present research.

Jointless Plain Concrete.—Test Sections 1A and 2A, 1,357 and 1,353 ft long, respectively, are plain concrete pavements without transverse joints. This basic type was included in the study because of existing evidence that similar pavements had rendered long and satisfactory service on many early-constructed miles of road in Illinois. It was considered important to investigate behavior of this type of pavement under heavier traffic densities and axle loads. Evaluation of its performance in comparison with that of other pavement types was contemplated as a possible means of developing a standby nonreinforced type of design suitable at least for periods of steel shortage.

Short-Panel Plain Concrete (without load-transfer devices).—Test Sections 1B and 2B, having dummy-groove contraction joints, are of a design not previously used on regular construction projects in Illinois, though sometimes used by other highway agencies. These sections are 1,337 and 1,353 ft long, respectively. The 20-ft panel lengths are without load-transfer devices. The sections are departures in these two respects from the main features of most of the other test sections where the standard panel length is 100 ft and dowel assemblies are present at contraction joints. This basic short-panel-length design, without steel for reinforcement or load transfer, is a second possibility being investigated for use during periods of steel shortage.

Sawed Contraction Joints.—Test Sections 3 and 6, 991 and 1,001 ft long, are of the Illinois standard design of 1951 except that the transverse joints were formed by sawing the hardened concrete. These two sections are similar except for the methods of curing. Section 3 was cured by the use of impermeable paper, whereas Section 6 was cured by the use of wetted burlap.

Dummy-Groove Contraction Joints.—Test Sections 4 and 5, 3,206 and 3,203 ft long, employ dummy-groove transverse joints in place of the metal-plate joints, but otherwise conform to the 1951 standard Illinois design. Section 4 differs from Section 5 in that an effort was made to abrade the edged faces of the joints of Section 4 prior to sealing.

Rubber-Asphalt Joint Sealing Compounds.—Test Section 7 varies from 1951 standard Illinois design only in the final treatment of the metal-plate contraction joints included within its 3,576-ft length. All of the contraction joints received abrading treatment, and 19 of the joints, including one nonabraded construction joint, were sealed with a hot-applied rubber-asphalt compound, whereas the remaining 17 contraction joints were sealed with a ready-mixed, cold-applied rubber-asphalt compound. Contrary to other test features, provision was not made for comparative evaluation between the rubber-asphalt compounds and the Illinois standard asphalt filler of PAF grades.

Abrading Opposing Vertical Faces of Contraction Joints.—Tendencies have been observed in the past in Illinois for loss of seal at joints to result not only from rupture of the sealing material and loss of adhesion between concrete and the sealing material, but also from separation between thin exterior films of mortar and the main body of the concrete, with bond apparently preserved between the contact face of the mortar film and the sealing material. Removal of the mortar film was the exploratory objective of the abrading process with improvement of adhesion as the ultimate objective. Dummy-groove joints of Section 4 and metal-plate joints of Section 7 were abraded.

Unedged Joints.—The section of pavement between Stations 39+16 and 90+97 and originally identified as Section X was to be established as a control section and was not intended to include experimental features when the research was initially planned. However, a statewide change in construction procedure, which eliminated hand edging of the metal-plate transverse joints in an attempt to improve riding quality, resulted in a change of plans for this section. It was decided that this section would afford an opportunity to compare the service behavior of edged and unedged metal-plate joints, and this purpose was accordingly added to the research program. Sixteen metal-plate contraction joints included within the section limits were finished with a hand edger in conformity with the standard practice earlier prevailing. The remaining 36 metal-plate joints were left unedged in accordance with the change in Illinois standard adopted about the same time as the research of Section 110X-5 got under way. The section having unedged joints is now identified as Section XA, and the section with the originally planned edged joints is identified as Section XB.

Pavement details of the individual test sections are shown in Figures 2 and 3. The contents of these figures require no special explanation.

SOILS

The test project traverses an area of dark-colored prairie soil where the undisturbed terrain is nearly level to gently sloping. Soil of the area is developed from thick to moderately thick loess and is medium-textured with moderately permeable subsoil. The preconstruction soil survey showed subsoils ranging from silty clay loam to silty clay over a layer of silt, the latter at depths varying from 4½ to 6 ft below the ground surface. The depth of borrow areas on the northern end of the project was limited to 4½ ft due to the silty nature of soil below that horizon. Soils are of A-4 and A-6 group classifications and are generally considered to be susceptible to pumping. A more detailed discussion of the subgrade soils is presented later.

TRAFFIC

US 66 is the main highway between Chicago and St. Louis and its traffic is one of the highest volumes of any rural

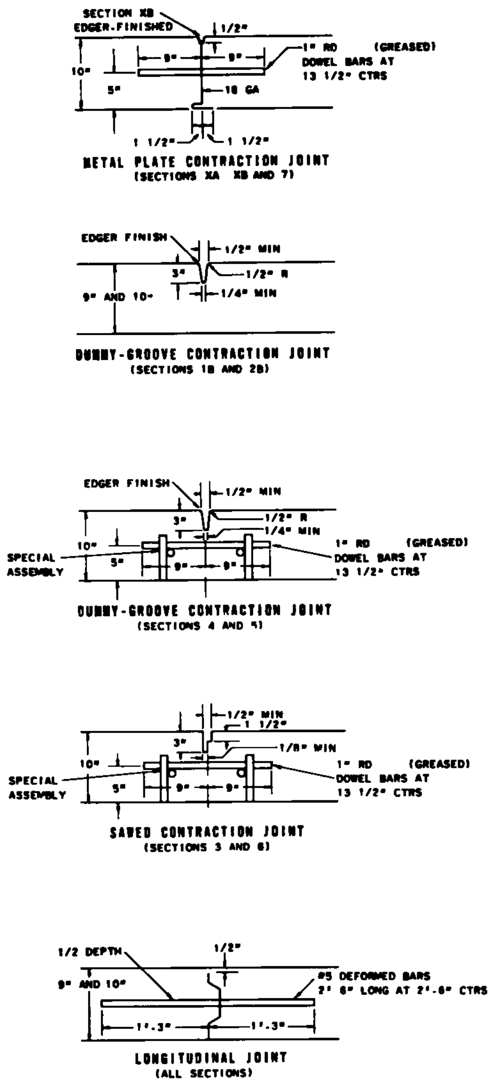


Figure 3. Details of contraction joints and longitudinal joint.

road in Illinois. Axle loads due to truck flow are likewise in the higher ranges of intercity routes. Traffic surveys in 1950 showed that the annual average daily volume carried by the two-lane pavement, at the location of Section 110X-5, amounted to 9,100 vehicles, of which 1,750 were trucks or other commercial types. The latter figure includes 805 tractor-truck semitrailer and full-trailer combinations. Traffic surveys made in 1953, about 1 yr after completion of the test project, showed that US 66, between the south junction of Springfield City US 66 and the Lake Springfield Bridge, now a divided, four-lane highway, served an average daily volume of 9,750 vehicles of all types, including 2,010 commercial vehicles with a further breakdown of 1,080 combinations. A survey of 1959 traffic shows that annual average daily volumes had risen to 12,600 total vehicles inclusive of 2,200 commercial types, including 1,400 combinations. The division between northbound and southbound traffic is approximately equal.

A permanent truck-weight station is located on US 66 approximately 35 mi south of the test project. It is believed that weight data recorded at this station are reasonably representative of heavy hauling over the test project. Although traffic volumes are considerably higher at the location of the test project than they are in the vicinity of the truck-weight station, classified counts at the two locations show that the heavier vehicles generally comprise through truck movements. It is therefore probable that

TABLE 2

TRUCK AXLE-WEIGHT AND VOLUME DATA

Weight Groups (pounds)	Daily Axle Loadings at Weight Station 35 Miles South of Experimental Project 1/			
	1953 Annual Average		1957 Summer Average	
	(number)	(percent)	(number)	(percent)
	Truck Single Axles			
Under 8,000	2,720	58	2,830	59
8,000 - 11,999	440	10	750	15
12,000 - 14,999	540	12	440	9
15,000 - 17,999	900	19	730	15
18,000 and Over	70	1	90	2
Total	4,670	100	4,840	100
	Truck Tandem Axles			
Under 16,000	200	19	470	26
16,000 - 19,999	90	9	170	9
20,000 - 23,999	140	14	190	11
24,000 - 27,999	200	19	300	17
28,000 - 31,999	350	34	600	33
32,000 and Over	50	5	70	4
Total	1,030	100	1,800	100

Location	Total Daily Vehicular Traffic	
	1953 Annual Average	1957 Summer Average
At Weight Station	6,100	8,300
At Test Project	9,750	16,200

1/ The numbers of axles at the test project in the heavier weight-groups are believed to be about the same as shown for the weight-station location. Northbound and southbound loadings are believed to be about equal.

heavy-truck axle loads are numerically approximately equal at the two locations.

With recognition of the foregoing explanation, the data of Table 2 are presented as an indication of the severity of axle loads to which the test sections have been subjected. Figures presented are those derived for the weight-station location, and they are without alteration to reflect differences in traffic volumes at the two locations. Appropriate 24-hr total traffic volumes over the test project, and at the weight station are given in the table to indicate differences in magnitudes of total-traffic flow.

CONSTRUCTION

The contract for the construction of Section 110X-5 was awarded in July 1951 and construction was started soon thereafter. Most of the grading, culvert construction, and part of the placing of the granular subbase was completed during the 1951 construction season. After a winter shutdown, work was resumed in the spring of 1952. Actual paving of the test sections was begun on May 21, and completed on July 25, 1952.

The new pavement was opened to traffic on July 22 and carried two-way traffic for about three weeks while the old pavement was being widened and resurfaced. Since August 9, 1952 the test sections have carried only the southbound traffic.

Grading

An embankment cross-section prevails through most of the project length because the finished grade of the pavement is generally from 1 to 3 ft above the relatively flat natural ground surface. Near the south end, a cut having a maximum depth of 5 to 6 ft extends about 1,000 ft in length, and terminates at a short, heavy fill (maximum height 14 ft) near Lake Springfield bridge. These are the only cuts and fills of consequence on the project.

The Illinois standard specifications, which governed the compaction of embankment for this project, required that construction methods be employed to secure not less than 90 percent of the maximum density shown on the wet-weight curve determined by the AASHO standard density test. The standard specifications further required that the moisture content of the material being placed should not exceed 110 percent of wet optimum.

Soil samples were secured from the compacted earth underneath the subbase and in-place density tests were made at 30 locations immediately ahead of paving operations. The samples, which were taken between Stations 91+00 and 264+65, represent the compacted earth to a depth of 6 in. The subbase material was sampled, and in-place subbase densities were determined, at the same locations. Test results of samples of the compacted earth are given in Table 3. These results substantiate those of the earlier soil tests made prior to construction. The soils are predominately dark colored silty clays of AASHO classification A-4(8) and A-6(8 to 12). Plasticity indexes range from 6 to 21 with a mean value of approximately 15. It is noteworthy that the uniformity of soil in this area is a circumstance favorable to the research that has been undertaken.

Field-density measurements of the compacted subgrade soil were made by the sand-hole method (AASHO Designation T-147-49) at the locations where the 30 samples were taken for which data are given in Table 3. Results in these field-density tests appear in Table 4 in combination with laboratory test data. Moisture-density tests were performed in the laboratory on nine of the 30 samples, these nine being chosen as representative of the range of the entire group of samples on the basis of grain size and Atterberg limits. Results of tests of the nine samples appear in Table 4. It will be seen from the table that relative densities are indicated to range from 92.5 to 105.8 percent, with a mean value of 98.2 percent on the wet-curve basis. The densities on the dry-curve basis range from 88.1 to 105.6 percent, with a mean value of 96.1 percent. The field water content of most of the samples was generally higher than optimum. The soil samples were taken and the in-place densities determined immediately prior to paving operations in the late spring of 1952, several months after the major portion of the embankment had been placed. The subbase was in place at the time of sampling and testing of the earth subgrade.

TABLE 3

PHYSICAL CHARACTERISTICS AND CLASSIFICATION OF SUBGRADE SOIL SAMPLES 1/

Sample Number	Station	Color	Soil Type	Group Classification	Mechanical Analysis Material Passing Sieves					Grain Size			Atterberg Limits		
					No. 10 (percent)	No. 20 (percent)	No. 40 (percent)	No. 100 (percent)	No. 200 (percent)	Sand 2.0 to 0.05mm (percent)	Silt 0.05 to 0.005mm (percent)	Clay < 0.005mm (percent)	LLL	LPL	PI
1	264+65	Brown	SiC	A-4(8)	-	100	99	96	90	10	60	30	24	18	6
2	263+50	Gray-Br.	SiC	A-6(9)	100	99	98	95	94	6	55	39	31	18	13
3	263+00	Gray-Br.	SiC	A-6(12)	100	99	98	95	95	6	56	38	36	17	19
4	262+00	Black	SiC	A-4(8)	100	99	98	96	95	5	60	35	31	21	10
5	261+00	Black	SiC	A-6(9)	100	99	98	96	95	5	60	35	32	19	13
6	260+00	Black	SiC	A-6(10)	-	100	99	97	97	5	56	39	34	18	16
7	259+00	Black	SiC	A-6(10)	100	99	97	94	94	6	57	37	35	19	16
8	258+00	Black	SiC	A-6(12)	100	99	98	96	96	9	52	39	38	18	20
9	264+90	Brown	SiCL	A-4(8)	100	98	97	94	93	10	61	29	27	19	8
10	257+00	Br.-Black	SiC	A-6(11)	-	100	99	97	97	5	59	36	35	17	18
11	256+00	Gr.-Brown	SiC	A-6(12)	100	99	98	96	95	7	57	36	38	18	20
12	255+00	Gr.-Brown	SiC	A-6(11)	100	98	96	93	93	10	53	37	37	19	18
13	254+00	Black	SiC	A-6(12)	100	99	99	98	97	6	58	36	39	20	19
14	250+00	Black	SiC	A-4(8)	-	100	99	96	96	9	61	30	27	17	10
15	244+00	Br.-Black	SiC	A-6(10)	100	99	98	96	95	6	55	39	37	21	16
16	234+50	Black	SiC	A-6(9)	-	100	98	97	97	7	56	37	34	22	12
17	220+00	Br.-Black	SiC	A-6(9)	100	99	99	97	97	11	59	30	34	22	12
18	182+00	Br.-Black	SiC	A-6(9)	100	98	97	95	94	8	52	40	35	23	12
19	168+00	Br.-Black	SiC	A-6(9)	100	99	98	96	96	7	53	40	36	23	13
20	164+00	Br.-Black	SiC	A-6(10)	-	100	99	98	97	5	57	38	36	22	14
21	160+00	Br.-Black	SiC	A-6(11)	-	100	99	98	98	4	57	39	38	20	18
22	155+00	Black	SiC	A-6(10)	-	100	99	98	98	7	54	39	37	21	16
23	150+00	Black	SiC	A-6(10)	100	99	96	93	92	15	50	35	34	20	14
24	145+50	Black	SiC	A-6(10)	100	99	98	96	96	9	52	39	37	22	15
25	132+50	Black	SiC	A-6(12)	100	97	93	88	87	13	51	36	39	18	21
26	118+25	Black	Clay	A-6(10)	100	99	96	91	91	15	48	37	39	23	16
27	113+00	Black	SiC	A-6(9)	100	99	97	94	93	11	52	37	35	23	12
28	106+00	Black	SiC	A-6(10)	100	99	99	97	96	6	54	40	38	23	15
29	98+00	Black	SiC	A-6(8)	-	100	99	97	96	6	57	37	32	21	11
30	91+00	Black	SiC	A-6(11)	100	99	98	95	94	7	56	37	40	22	18

1/ Samples include only materials in top of subgrade at average depth range of 0 to 6 inches.

TABLE 4

MOISTURE-DENSITY RELATIONSHIPS OF SUBGRADE SOIL SAMPLES 1/

Sample Number	Station	Soil Type	Field Water Content	Wet Curve Basis					Dry Curve Basis				
				Field Density	Optimum Moisture	Maximum Density	Relative Density	Relative Moisture Content	Field Density	Optimum Moisture	Maximum Density	Relative Density	Relative Moisture Content
			(percent)	(p.c.f.)	(percent)	(p.c.f.)	(percent)	(percent)	(p.c.f.)	(percent)	(p.c.f.)	(percent)	(percent)
1 2/	264+65	SiC	20.6	120.5	15.8	130.3	92.5	130.4	99.9	14.4	113.4	88.1	143.3
2 2/	263+50	SiC	23.3	125.5	19.2	124.3	101.0	121.4	101.8	17.3	105.7	96.3	134.7
3	263+00	SiC	24.0	126.3	(20.2)	(123.0)	102.7	118.8	101.9	(18.8)	(103.0)	98.9	127.7
4 2/	262+00	SiC	13.6	125.5	19.0	123.7	101.5	71.6	110.5	18.1	104.6	105.6	75.1
5 2/	261+00	SiC	23.1	126.0	20.2	124.2	101.4	114.4	102.4	17.8	104.8	97.7	129.8
6	260+00	SiC	23.7	126.0	(20.2)	(123.0)	102.4	117.3	101.9	(18.8)	(103.0)	98.9	126.1
7	259+00	SiC	24.9	122.4	(20.2)	(124.2)	98.6	123.3	98.0	(17.8)	(104.8)	93.5	139.9
8 2/	258+00	SiC	21.9	114.6	20.5	123.2	92.6	106.8	94.0	19.4	102.8	91.4	112.9
9	264+90	SiCL	20.2	121.2	(15.8)	(130.3)	93.0	127.8	100.8	(14.4)	(113.4)	88.9	140.3
10	257+00	SiC	17.7	121.5	(20.2)	(123.0)	98.8	87.6	103.2	(18.8)	(103.0)	100.2	94.1
11	256+00	SiC	21.2	116.0	(20.5)	(123.2)	93.8	103.4	95.7	(19.4)	(102.8)	93.1	109.3
12	255+00	SiC	16.5	119.7	(20.2)	(123.0)	97.3	81.7	102.7	(18.8)	(103.0)	99.7	87.8
13	254+00	SiC	17.3	120.0	(20.2)	(123.0)	97.6	85.6	102.3	(18.8)	(103.0)	99.3	92.0
14	250+00	SiC	20.0	126.2	(19.0)	(123.7)	102.0	105.3	105.2	(18.1)	(104.6)	100.6	110.5
15	244+00	SiC	28.0	116.5	(20.2)	(123.0)	94.7	138.6	91.0	(18.8)	(103.0)	88.3	148.9
16	234+50	SiC	14.6	122.5	(20.9)	(123.1)	99.5	69.9	106.9	(20.1)	(102.3)	104.5	72.6
17	220+00	SiC	26.8	117.7	(20.9)	(123.1)	95.6	128.2	92.8	(20.1)	(102.3)	90.7	133.3
18 2/	182+00	SiC	27.9	116.9	20.9	123.1	95.0	133.5	91.4	20.1	102.3	89.3	138.8
19	168+00	SiC	27.0	115.3	(20.9)	(123.1)	93.7	129.2	90.8	(20.1)	(102.3)	88.8	134.3
20	164+00	SiC	22.6	121.5	(23.7)	(119.8)	101.4	95.4	99.1	(21.9)	(97.8)	101.3	103.2
21 2/	160+00	SiC	23.4	120.9	20.2	123.0	98.3	115.8	98.0	18.8	103.0	95.1	124.5
22	155+00	SiC	27.0	117.9	(23.7)	(119.8)	98.4	113.9	92.8	(21.9)	(97.8)	94.9	123.3
23	150+00	SiC	26.4	118.3	(23.7)	(119.8)	98.7	111.4	93.6	(21.9)	(97.8)	95.7	120.5
24 2/	145+50	SiC	26.7	115.7	23.7	119.8	96.6	112.7	91.3	21.9	97.8	93.4	121.9
25	132+50	SiC	25.4	126.6	(20.5)	(123.2)	102.8	123.9	101.0	(19.4)	(102.8)	98.2	130.9
26	118+25	Clay	24.2	115.2	(23.7)	(119.8)	96.2	102.1	92.8	(21.9)	(97.8)	94.9	110.5
27	113+00	SiC	18.6	118.0	(20.9)	(123.1)	95.9	89.0	99.5	(20.1)	(102.3)	97.3	92.5
28	106+00	SiC	24.7	126.8	(23.7)	(119.8)	105.8	104.2	101.7	(21.9)	(97.8)	104.0	112.8
29 2/	98+00	SiC	20.3	123.0	20.6	122.0	100.8	98.5	102.2	19.9	102.0	100.2	102.0
30	91+00	SiC	25.6	120.3	(20.2)	(123.0)	97.8	126.7	95.8	(18.8)	(103.0)	93.0	136.2

1/ Samples taken at average depths of 0 to 6 in. below subbase.

2/ Moisture-density tests were performed in laboratory on these 9 samples chosen as representative of the entire group of samples on the basis of grain-size analysis and Atterberg limits. Values enclosed in parentheses are from laboratory test of one of the nine samples most nearly representative of the respective sample.

Subbase

A trench-type, undrained subbase, extending 1 ft beyond the pavement at each edge and having a design compacted thickness of 6 in., was used throughout the experimental project. The Illinois specifications for granular subbase at the time this project was constructed required that the material be uniform, and conform to the following gradation:

Percent Passing Sieves (Square Openings)			
3 in.	No. 4	No. 50	No. 200
100	45-90	5-25	5-10

There was no specification regarding the plasticity of the material.

The information regarding tests of granular subbase (Tables 5 and 6) related to 30 samples of such material, each sample taken and each density test made at the same location as the correspondingly numbered sample of subgrade soil given in Table 4.

Subbase material placed in the fall of 1951 extended from Station 113+00 to Station 121+00 and from Station 167+00 to Station 265+01. Two materials of somewhat different gradation were used in these areas. A bank-run gravel was first placed in a lift of varying thickness but averaging about 4 in. On top of this was placed a lift of coarse sand to complete the specified 6-in. total thickness. Subbase for the remainder of the experimental area was placed in 1952. This consisted of a single 6-in. course of bank-run gravel, spread and compacted immediately ahead of the paving operation.

At the time the samples reported in Tables 5 and 6 were obtained, the stratification of the two layers of slightly different material placed in 1951 could be detected in only a few instances. Presumably, considerable mixing of the two materials took place during the construction operation.

All samples of subbase material taken in areas of 1951 construction were nonplastic. Most, but not all, of the samples taken in areas of 1952 construction showed a small amount of plasticity. Physical characteristics determined for the samples are given in Table 5. All samples classified as either A-1-a(0) or A-1-b(0) sandy gravels.

The results of the field-density and field-moisture determinations, and the moisture-density relationships for the subbase samples are given in Table 6. Actual laboratory density tests were made on composite samples consisting of three or more of the field samples. Six groupings of 29 samples were made based on similarities of gradation and Atterberg-limit values. The relative densities averaged 91.5 percent on the wet-curve basis and 92.2 percent on the dry-curve basis. Relative densities were not determined for locations where two layers of material were clearly visible because the field work did not include a separate determination of the density of each layer.

The material having some plasticity appeared to compact more easily and to produce a more stable subbase with respect to supporting construction equipment than did the nonplastic material. This latter material remained loose on top even after several passes of the rubber-tired rollers.

Paving Procedures and Curing

Batch trucks traveling on the shoulder transported materials to the paver which was also operated on the shoulder. Paving progressed full width from south to north. The paving train for the period from May 21, 1952 through June 3, 1952 consisted of one paver followed by a mechanical concrete spreader, a finishing machine, and a mechanical longitudinal float. On June 4, 1952 a second paver and mechanical spreader were added to the paving train. After the passage of the mechanized equipment, hand straightedging, belting and brooming completed the surface-finishing operation except for a length of 3,628 ft of pavement at the north end where a burlap-drag finish was substituted for the broom finish. The burlap-drag finish was produced by two passes of wetted burlap over the surface of the concrete following belting. The change to the burlap drag at the north end of the project came about as the result of a general change in construction procedure throughout the state.

TABLE 5

PHYSICAL CHARACTERISTICS AND CLASSIFICATION OF GRANULAR SUBBASE SAMPLES 1/

Sample Number	Station	Subbase Placed 2/	Material Type	Group Classification	Mechanical Analysis Passing Sieves 3"	No. 4	No. 50	No. 200	Atterberg Limits LLL	LPL	PI
				Specifications	100 (percent)	45-90 (percent)	5-25 (percent)	5-10 (percent)			
1	264+65	F 51	Sandy Gravel	A-1-a(0)	100	68	18	6			NP
2	263+50	F 51	"	A-1-b(0)	100	72	14	6			NP
3	263+00	F 51	"	A-1-b(0)	100	77	18	5			NP
4	262+00	F 51	"	A-1-b(0)	100	89	18	5			NP
5	261+00	F 51	"	A-1-b(0)	100	86	17	4			NP
6	260+00	F 51	"	A-1-b(0)	100	77	17	5			NP
7	259+00	F 51	"	A-1-b(0)	100	69	16	6			NP
8	258+00	F 51	"	A-1-a(0)	100	73	19	6			NP
9	264+90	F 51	"	A-1-b(0)	100	85	16	4			NP
10	257+00	F 51	"	A-1-b(0)	100	77	19	6			NP
11	256+00	F 51	"	A-1-b(0)	100	76	20	7			NP
12	255+00	F 51	"	A-1-b(0)	100	76	21	10			NP
13	254+00	F 51	"	A-1-b(0)	100	78	19	7			NP
14	250+00	F 51	"	A-1-b(0)	100	91	25	13			NP
15	244+00	F 51	"	A-1-b(0)	100	88	28	15			NP
16	234+50	F 51	"	A-1-a(0)	100	72	12	5			NP
17	220+00	F 51	"	A-1-b(0)	100	71	18	8			NP
18	182+00	F 51	"	A-1-b(0)	100	73	19	11			NP
19	168+00	F 51	"	A-1-a(0)	100	66	15	8			NP
20	164+00	S 52	"	A-1-b(0)	100	87	25	16	16	11	5
21	160+00	S 52	"	A-1-b(0)	100	85	21	13	16	11	5
22	155+00	S 52	"	A-1-b(0)	100	85	22	13	16	12	4
23	150+00	S 52	"	A-1-b(0)	100	85	26	17	16	14	2
24	145+50	S 52	"	A-1-b(0)	100	89	19	12			NP
25	132+50	S 52	"	A-1-a(0)	100	62	13	6			NP
26	118+25	F 51	"	A-1-b(0)	100	87	19	12			NP
27	113+00	F 51	"	A-1-b(0)	100	92	26	13			NP
28	106+00	S 52	"	A-1-b(0)	100	85	18	10			NP
29	98+00	S 52	"	A-1-b(0)	100	86	24	15	16	11	5
30	91+00	S 52	"	A-1-b(0)	100	88	26	15	17	13	4

1/ Samples include subbase material from an average depth of 0 to 6 in. below the bottom of the pavement

2/ F 51 - indicates fall of 1951. S 52 - indicates summer of 1952.

TABLE 6

MOISTURE-DENSITY RELATIONSHIPS OF GRANULAR SUBBASE SAMPLES 1/

Sample Number	Station	Material Type	Field Water Content	Wet Curve Basis					Dry Curve Basis				
				Field Density	Optimum Moisture	Maximum Density	Relative Density	Relative Moisture Content	Field Density	Optimum Moisture	Maximum Density	Relative Density	Relative Moisture Content
			(percent)	(p.c.f.)	(percent)	(p.c.f.)	(percent)	(percent)	(p.c.f.)	(percent)	(p.c.f.)	(percent)	(percent)
1	264+65	Sandy Gravel	7.0	89.9					84.0				
2	263+50	"	7.4	90.4	9.1	138.4	65.3	81.3	84.2	8.9	127.1	66.2	83.1
3	263+00	"	5.5	126.3	9.1	138.4	91.3	60.4	119.7	8.9	127.1	94.2	61.8
4	262+00	"	9.7	124.7					113.7				
5	261+00	"	13.9	124.1					109.0				
6	260+00	"	6.0	128.0	9.1	138.4	92.5	65.9	120.8	8.9	127.1	95.0	67.4
7	259+00	"	9.7	129.4					118.0				
8	258+00	"	7.1	126.3	9.6	141.6	89.2	74.0	117.9	9.1	129.6	91.0	78.0
9	264+90	"	17.2	125.1					106.7				
10	257+00	"	9.6	134.6	9.1	139.4	96.6	105.5	122.8	8.8	127.7	96.2	109.1
11	256+00	"	12.3	140.0	9.1	139.4	100.4	135.2	124.7	8.8	127.7	97.7	139.8
12	255+00	"	6.0	143.6	9.1	139.4	103.0	65.9	135.5	8.8	127.7	106.1	68.2
13	254+00	"	7.9	85.7					79.4				
14	250+00	"	10.6	140.5	9.8	137.1	102.5	108.2	127.0	9.5	125.0	101.6	111.6
15	244+00	"	7.3	130.6	9.8	137.1	95.3	74.5	121.7	9.5	125.0	97.4	76.8
16	234+50	"	10.3	133.3	9.6	141.6	94.1	107.3	120.9	9.1	129.6	93.3	113.2
17	220+00	"	7.9	140.6	9.8	137.1	102.6	80.6	130.3	9.5	125.0	104.2	83.2
18	182+00	"	4.2	128.6	9.0	137.2	93.7	46.7	123.4	8.7	126.2	97.8	48.3
19	168+00	"	5.7	131.1	9.6	141.6	92.6	59.4	124.0	9.1	129.6	95.7	62.6
20	164+00	"	7.5	127.2	9.0	137.2	92.7	83.3	118.3	8.7	126.2	93.7	86.2
21	160+00	"	5.3	117.0	9.0	137.2	85.3	58.9	111.1	8.7	126.2	88.0	60.9
22	155+00	"	7.3	128.6	9.0	137.2	93.7	81.1	119.9	8.7	126.2	95.0	83.9
23	150+00	"	4.7	122.6	9.0	137.2	89.3	52.2	117.1	8.7	126.2	92.8	54.0
24	145+50	"	7.3	108.9					101.5				
25	132+50	"	7.5	128.4	9.6	141.6	90.7	78.1	119.4	9.1	129.6	92.1	82.4
26	118+25	"	11.0	131.5	9.8	137.1	95.9	112.2	118.5	9.5	125.0	94.8	115.8
27	113+00	"	9.2	126.4	9.8	137.1	92.2	93.9	115.8	9.5	125.0	92.6	96.8
28	106+00	"	8.6	124.8	9.8	137.1	91.0	87.8	114.9	9.5	125.0	91.9	90.5
29	98+00	"	8.6	130.3	9.0	137.2	95.0	95.6	120.0	8.7	126.2	95.1	98.9
30	91+00	"	11.1	136.9	9.0	137.2	99.8	123.3	123.2	8.7	126.2	97.6	127.6

1/ Laboratory moisture-density tests were performed on combinations of three or more samples grouped on the basis of similarities of gradation and Atterberg limits. Application of the laboratory test results has been omitted in cases of samples 1, 4, 5, 7, 9, 13 and 24 because, at the time of field sampling, the subbase material showed two distinct courses which were not tested separately.

Where mesh reinforcement was used, the concrete was placed in two lifts. When one mixer and spreader were in use, the first lift was placed and struck off about 2½ in. below the top of the forms for a distance of 50 to 60 ft ahead of the finishing machine. The mixer and spreader were then backed while the mesh was being installed, and the top lift of concrete was placed and spread. With two mixers and spreaders in use, the bottom lift was placed by the lead mixer and struck off by the lead spreader, and the top lift was placed with the second mixer and struck off with the second spreader. This latter procedure was followed in general for certain test sections where mesh was omitted, but the lifts placed were more nearly equal in thickness, permitting the second mixer to operate nearer its full capacity.

The concrete was consolidated from the surface by a pan-type vibrator mounted at the rear of the mechanical spreader.

In paving adjacent to the full-depth metal-plate contraction joints, the concrete was placed carefully on each side of the metal plate and hand-spaded into place to hold the plate in correct alignment. The spreader was raised over the plate and the pins holding the plate cap were removed after the passage of the spreader over the joint. The front screed of the finishing machine was also raised over the plate to prevent displacement. At locations where the joints were to be edged, the plate cap was left on until after the passage of the longitudinal float. At locations where edging was to be omitted, the plate cap was removed ahead of the longitudinal float.

In paving over the basket assemblies holding the load-transfer dowels, at locations of dummy-groove and sawed joints, the concrete was placed directly over the basket assemblies and no special precautions were required for further spreading and finishing operations. The dummy-groove joints were formed by forcing a steel bar having a T-shaped cross-section into the fresh concrete. These joints were hand-finished with an edging tool after the concrete had attained proper stiffness. The locations of the center of the dowels were marked on the forms and later in the fresh concrete at each pavement edge to accurately position the dummy-groove and sawed contraction joints.

Vibrators operating from the lead spreader were used to consolidate the concrete around assemblies of dowel bars at transverse joints and tie bars of the longitudinal center joint, and along the forms.

The impermeable-paper method of curing was used on all but one of the test sections. The pavement of the section on which impermeable paper was not used was cured with wetted burlap. Full-width paper was placed on the pavement after the surface moisture had disappeared and the concrete had stiffened sufficiently to support the paper without being marred. Extra strips, 2 ft wide, were placed along each edge and inserted under the full-width paper. These were pulled down over the pavement edges after the forms were removed. The paper remained in place for a minimum period of 72 hours after placement of the concrete. On the section where wetted burlap was substituted for the paper (Section 6), two thicknesses of burlap were applied and saturated as soon as the concrete had stiffened sufficiently to permit such application without marring the surface. The burlap was kept thoroughly saturated by periodic sprinkling, and remained in place for a minimum period of 72 hours.

On Sections 3 and 6 having sawed joints spaced at 100-ft intervals, the initial saw cut was made on the day following paving (two joints were sawed four days following paving) by one pass of a single-bladed machine. A diamond-type saw blade was used in the cutting operation and cut a kerf of about ⅛ in. width. Initial cuts were made to depths of 2¼ and 3 in. The saw groove was widened at a later date by an additional parallel cut to a 1½ in. depth. The final grooves were about ½ in. in width.

Although the sawing machine that was used was somewhat primitive when compared with those used in present-day construction, the operation was the same as that in general use today and no particular problems were involved.

Mix Design, Paving Control, Tests and Test Specimens

The coarse aggregates for the pavement consisted of crushed limestone, having a specific gravity of 2.65. The typical gradation for each size was as follows:

	Percent Passing Sieves (Square Openings)					
	2½ in.	2 in.	1½ in.	1 in.	½ in.	No. 4
Size A	100	97	52	6	2	
Size B				100	40	3

The fine aggregate consisted of natural sand, having a specific gravity of 2.65. The typical gradation was as follows:

Percent Passing Sieves (Square Openings)				
⅜ in.	No. 4	No. 8	No. 16	No. 50
100	97	83	72	18

Type 1A cement was used throughout.

The concrete mix was designed using the mortar-voids theory as normally applied by Illinois. The cement factor ranged from 1.42 to 1.45 bbl of cement per cu yd of concrete, with 1.44 being used for the major portion of the test pavement. Approximately 5.4 gallons of water per sack of cement provided satisfactory workability. The mix that may be considered as typical was as follows:

Cement	-	94 pounds
Sand	-	200 pounds
C.A. Size A	-	172 pounds
C.A. Size B	-	172 pounds
Water	-	5.4 gallons

Usual standard Illinois paving controls were used throughout the project.

Tests usually were made twice daily to determine the percentage of entrained air in the concrete. Sampling and testing were essentially the same as those of the Test for Air Content of Freshly Mixed Concrete by the Pressure Method—AASHO Designation T 152-53. The consistency of the freshly mixed concrete was measured by the standard slump test. The results of the air-content tests and the slump tests are reported in Table 7. The air content of the plastic concrete averaged 3.9 percent and slump averaged 3.1 in.

Fifty 6- by 6- by 30-in. beam specimens were made during the placing of the test pavement. Four beams were made per day during the first three days of paving. After the third day, two per day were made. Beams were cast from freshly mixed concrete representative of the mixture and covered with wetted burlap. As soon as the beam forms could be removed and the beams could be transported without damage, they were taken to a sand pit and cured in damp sand until tested for flexural strength. Testing was performed on a modified cantilever-type machine, employing the center-point method of loading. When four beams were being made per day, they were tested usually at ages of 3, 5, 7 and 14 days. When only two beams were made per day, testing was normally at the ages of 7 and 14 days. The results of 7- and 14-day beam tests are given in Table 7. The average modulus of rupture at age 14 days was 749 psi.

A total of 44 cores were drilled from the hardened concrete before the pavement was opened to traffic. These specimens were 4½ in. in diameter and are drilled primarily as a check on pavement thickness. The core lengths indicate that all pavement equaled or slightly exceeded the specified thicknesses. The cores were tested for compressive strength on July 17, 1952 at an average age of approximately 40 days. The locations at which the cores were drilled and the results of the tests are given in Table 8. The average value of compressive strength was 4,433 psi, and the maximum and minimum values were 6,434 and 3,489 psi, respectively.

RESEARCH PROCEDURE

It has been mentioned previously that the controls exercised over the construction operations differed in no way from usual Illinois practice on any normal construction project. Supervision and routine inspection were handled by the resident engineer and his assistants. Special observations to obtain information likely to be of aid in inter-

TABLE 7

PORTLAND CEMENT CONCRETE MIX CONTROL TEST DATA

Stations of Test Samples & Beam Specimens	Entrained Air	Slump	Modulus of Rupture 7 Days	Modulus of Rupture 14 Days
	(percent)	(inches)	(psi)	(psi)
262+15	4.9	5.0	613	585
253+00	4.7	2.0	549	841
246+00	4.7	3.3	643 1/	775
237+50	4.7	2.5	664 2/	809
227+50	4.3	2.9	689	858
217+00	3.0	4.0	626	700
208+25	3.7	2.8	709	794
198+00	3.7	2.5	649	740
183+80	4.3	3.5	598	755
170+50	3.2	3.0	625	694
160+20	3.1	3.5	600	721
147+40	3.5	3.5	631	781
132+80	3.9	3.0	690	751
131+16	3.2	3.0	561	723
126+10	3.2	2.8	615	714
112+50	3.9	3.0	760	733
96+00	4.5	3.0	648	773
80+25	3.8	3.2	585	719
63+75	4.2	3.5	649	735
58+40	4.4	3.0	723	812
51+75	3.9	3.5	580	685
44+48	4.2	3.0	655 3/	790
Average	3.9	3.1	639	749

1/ Tested at age 10 days. 2/ Tested at age 9 days 3/ Tested at age 5 days.

preting the results of research were made by a research engineer and in some instances by a materials engineer.

The only measurements entirely of a research nature that were made during construction were at special plugs installed in the concrete to furnish information on slab movements at joints. These are described in detail later.

Pavement Condition Surveys

Several periodic surveys of pavement behavior have been made since the construction of the test project. The first of these, which did not include Sections XA and XB, was completed immediately prior to the opening of the pavement to traffic on July 22, 1952. Follow-up surveys have been made once or twice a year since that date and have included Sections XA and XB.

The condition surveys have been made on foot, and the presence and location of such defects as have been found have been mapped. The only defects noted thus far have been cracks, spalls, faults, and shoulder holes. Because of the significance of spall in assessing the relative behavior of the experimental joint installations, the amount and degree of spall have been mapped in considerable detail.

The condition of the joint seal has also been observed and recorded during the condition surveys.

Measurement of Faults

A fault-measuring device (Fig. 4) which has been in use for several years in Illinois and which has been found to be superior to similar devices that have been employed

TABLE 8

COMPRESSIVE STRENGTH OF PAVEMENT CORES
(Approximate Average Age - 40 Days)

Station	Compressive Strength (psi)	Station	Compressive Strength (psi)
263+20	4587	141+35	3942
257+70	4354	138+40	4606
251+00	4134	132+60	4440
245+65	4585	126+40	6434
235+35	4856	121+00	4854
230+35	4662	115+90	5237
225+95	4263	109+75	4806
221+25	4186	105+64	4102
216+00	3489	103+65	4437
212+65	4122	102+00	3896
205+70	5163	98+15	5256
199+75	4874	94+30	4250
192+90	3721	90+65	3537
188+75	5123	86+90	4413
183+30	4154	84+15	4942
177+40	4288	79+85	4199
171+25	4005	74+95	4256
165+95	5170	70+35	4431
159+70	4334	65+45	5070
154+90	4134	64+15	3929
149+80	3918	60+85	4192
144+75	3918	44+00	3588
		Average, 44 cores	4433

previously was used to measure differential vertical slab displacement at the joints and cracks. This meter was developed after it was found that the devices previously used and supported by two legs on one side of the joint produced erroneous readings where the pavement surface at the joints was higher than the surrounding pavement—a condition which was found to be of frequent occurrence.

The two free-moving members, shown in Figure 4, are principal features that distinguish the new fault meter from earlier models with a single upright moving member. Whereas use of the earlier model required its placement on one slab with measurement made across the separation to the adjoining slab, the new device is placed astride of a joint or crack so that it rests on the two adjoining pavement slabs with supporting shoes approximately equidistant from the joint or crack being measured for faulting.

Fault meter surveys of the test sections have been made periodically since construction. Faults are measured in the wheel paths at all transverse joints and cracks.

Measurement of Slab Movements

Because a major phase of study under this research project relates to the design and performance of pavement slabs of different lengths, these being articulated by contraction joints of varying functional design in respect to slab-separation and load-transfer, it was decided to provide for making rather precise horizontal and vertical measurements at joints, and also at transverse cracks. Therefore, brass reference plugs were set at most of the joints, and soon after construction at most of the cracks that occurred. Cracking immediately following construction was confined to the unjointed sections.

The plugs are ⅜ in. in diameter by 1 in. long, each having a ⅛-in. diameter hole drilled at the center axis. These were installed in pairs at the joints and cracks where measurements were to be made. They were placed about 10 to 13 in. each side of joints and cracks and approximately 6 in. centerward from the (west) edge of the pavement.

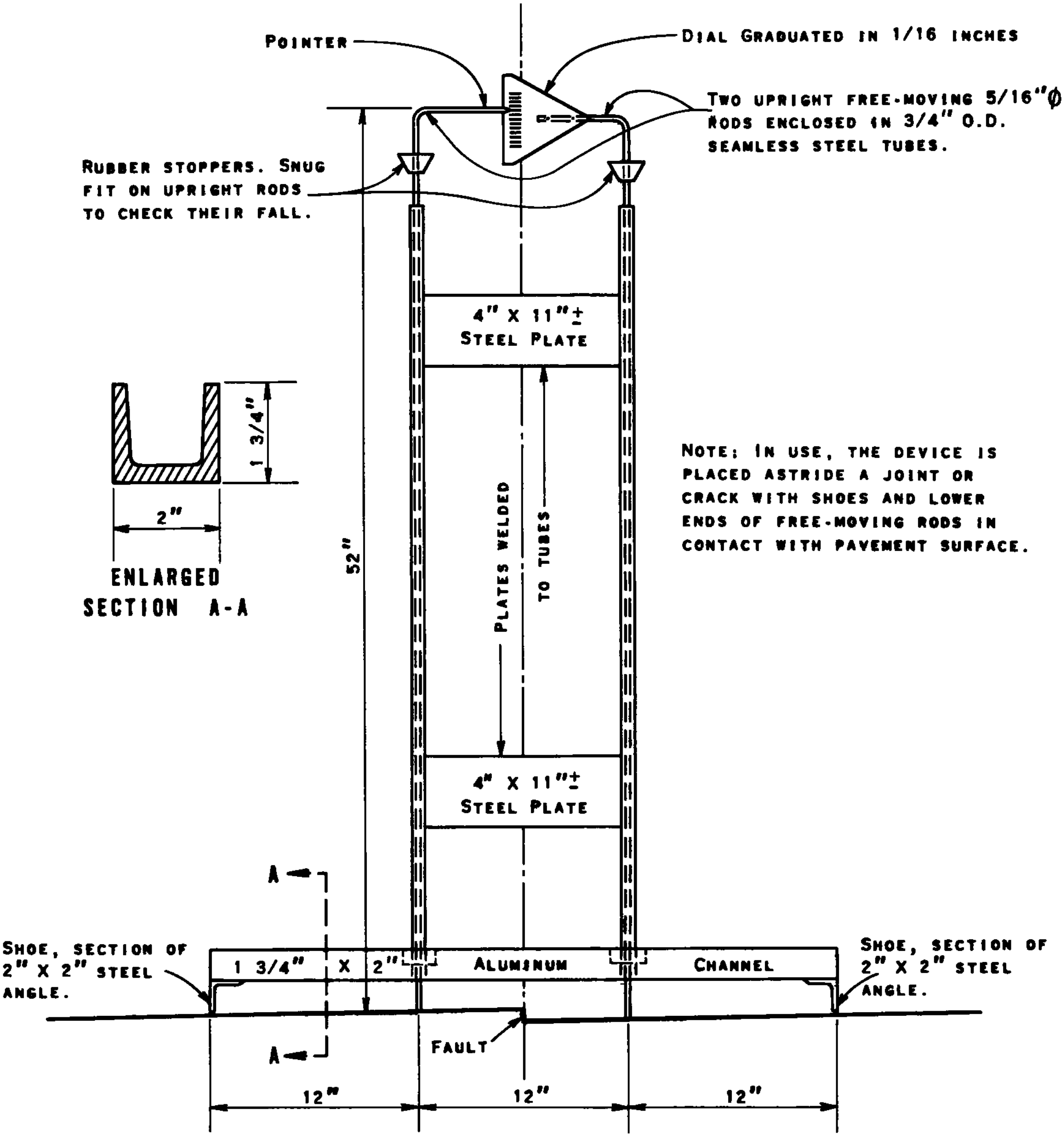


Figure 4. Mechanical principles of fault-measuring device.

Vertical placement is about ⅛ in. below the pavement surface for protection from snow-removal equipment. The reference plugs were installed in most instances in the plastic concrete. At the cracks, and at a few of the joints, they were installed in the hardened concrete using a star drill. Center holes of the plugs were closed against infiltering foreign material by copper rivets placed in the holes. Installations have been made at 172 transverse joints, and at 27 panel cracks.

Repetitive measurements have been made horizontally and vertically between the various sets of plugs using a specially-designed extensometer. By means of verniers, readings are made to 0.001 in. horizontally and to 0.01 in. vertically. Caliper points of the extensometer are cone-shaped, and are self-centering when placed in the ⅛-in. holes in the reference plugs. A level bubble is mounted on the extensometer, and the instrument is leveled at the time of reading. Recorded readings consist of the horizontal distance between plugs and the elevation differential of the tops of the plugs. Initial readings were made at each set of plugs as soon after installation as the concrete became firm. Horizontal readings indicate the opening and closing that takes place at the contraction joints and cracks, and the vertical readings are for correlation with faulting that may take place at the joints or cracks.

Following the initial recording of extensometer readings at the joints of each test section, an early series of readings was made at frequent but irregular intervals. Such early readings were continued until there was conclusive evidence of slab separation at each joint. Subsequent extensometer readings have been made at irregular intervals varying from 2½ to 17 months. Some of the longer intervals prevent much significance being derived from trends that can be established to date, but this may not be seriously disadvantageous in respect to long-term movements if the readings are continued at regular summer and winter intervals in the future.

Surface Smoothness

A road-smoothness indicator patterned after the Bureau of Public Roads' indicator (see "Standardizable Equipment for Evaluating Road Surface Roughness," Public Roads, Vol. 21, No. 12, February 1941) and constructed by the Illinois Division of Highways became available for use on the test pavement in July 1957.

Miscellaneous Research

As aids in understanding the test features a few comments regarding research procedure are as follows: (1) About one-half of the joints of Section 7 were sealed with a hot-application rubber-asphalt material, whereas the remaining joints of Section 7 and of all other test sections were sealed with a cold-applied rubber-asphalt material. It is noteworthy that, although the contract special provisions called for use of a double-boiler type of kettle for heating the hot-application compound, the manufacturer of the material, in personal charge of the use of his product on the test project, elected to forego the expense of providing the special equipment because of the small quantity of material to be used, finally amounting to about 1 cu ft. Application was therefore by a somewhat crude method, and it is not known whether test results were altered thereby. (2) A joint-grooving machine was used to abrade and remove surface mortar from the opposing vertical faces of some of the hand-edged contraction joints prior to sealing. The abrading force was transmitted by means of a high-speed revolving drum guided above the road surface along the joints. Star-shaped, nonpowered cutting wheels, positioned around the drum-periphery, served as the abrading medium. Staggering of cutters permitted simultaneous abrading of both faces of a joint. Spacer-washers provided adjustment of abrading width. Effectiveness of the machine was impaired by some spalling of joints and by not achieving complete continuity of abrasion. There was a resulting belief that the particular unit of equipment was not entirely suitable for the purpose for which it was used.

EVALUATION OF OBSERVATIONS

The test pavements have served satisfactorily during seven years of use under normal traffic. There has been no pumping, likewise no scaling. Few shoulder holes have been in evidence. Cracks are infrequent, and those occurring are principally of the transverse type, although a few infiltration cracks are present here and there. (The term "infiltration" is used here to identify cracks that begin at transverse joints and transverse cracks most frequently from 1 to 3 ft in from the outside edge of the pavement, and extend in a generally longitudinal direction a few inches to sometimes several feet.) Faulting at transverse joints and cracks up to the present time is generally slight. Spalls are frequent at the joints and cracks of certain test sections, but are in but a few instances more than superficial in nature.

Observations of the experimentation, in cases of some items, began prior to the close of construction, and have continued periodically for all items of research. Such observations are discussed under topic headings that follow.

Cracks

Results of the most recent crack survey, made in 1959, are given in Table 9. As stated previously the only crack types that have appeared are transverse cracks and "infiltration" cracks.

It will be noted in Table 9 that the transverse cracks that have occurred during the first seven years of pavement life are few in number, except in the two test sections of plain concrete built without transverse joints. Even in these sections the average panel lengths formed by the cracks are after seven years within the general range of the panel lengths of the reinforced sections constructed with joints at 100-ft intervals. The fact that the average uncracked slab length for the 9-in. thick nonjointed pavement is slightly greater than that for the 10-in. thick nonjointed pavement is not considered to be significant at this time.

Details regarding the progression of transverse cracking that has taken place in the nonjointed and nonreinforced pavement test sections are presented in Table 10. It will be noted from the table that the bulk of the cracking that has taken place in the first seven years of pavement life occurred during the first eight months.

TABLE 9

CRACKS IN PAVEMENT, SHOULDER HOLES AND PUMPING

(1959 Survey)

Test Section	Distinctive Design Features	Section Length (feet)	Transverse Cracks (number)	Average Joint and Crack Interval (feet)	Infiltration Cracks (number)	Shoulder Holes 1/ (number)	Pumping (number)
X A	10-in. reinforced concrete;unedged metal-plate joints at 100 ft	3620	0	93	3	0	
X B	10-in. reinforced concrete; edged metal-plate joints at 100 ft	1561	1	82	0	0	
1A	10-in. plain concrete; no joints	1357	17	75	1	0	
1B	10-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	1337	0	20	0	13	
2A	9-in. plain concrete; no joints	1353	15	85	3	5	None
2B	9-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	1353	0	20	0	23	
3	10-in. reinforced concrete; sawed joints at 100 ft	991	2	71	1	3	
4	10-in. reinforced concrete; dummy-groove joints at 100 ft.(abraded faces)	3206	3	89	4	7	
5	10-in. reinforced concrete; dummy-groove joints at 100 ft	3203	9	73	1	6	
6	10-in. reinforced concrete; sawed joints at 100 ft	1001	2	83	0	0	
7	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	3576	6	83	1	6	

1/ All shoulder holes adjacent to main travel lane

TABLE 10

TRANSVERSE CRACKS IN PLAIN CONCRETE
NONJOINTED SECTIONS AT VARIOUS AGES

Age of Pavement	Section 1A, 10-in. Thick 1/		Section 2A, 9-in. Thick 2/	
	Passing Lane	Travel Lane	Passing Lane	Travel Lane
3 days	1	1	0	0
4 days	2	2	1	1
5 days	2	2	3	3
7 days	2	2	6	6
6 weeks	5	5	7	7
7 weeks	7	7	7	7
8 weeks	7	7	10	10
8 months	15	15	11	11
3 years	15	15	11	12
5 years	15	15	14	16
7 years	16	17	14	16

1/ Length 1357 feet; 1 construction joint
2/ Length 1353 feet; 1 construction joint

Infiltration cracks that have developed to date are relatively few in number (Table 9). Those that have appeared are barely visible and are not considered to be especially significant as evidence of deterioration. If the present trend toward a concentration of these cracks on the sections with transverse joints and cracks at the longer intervals continues, these will be considered to have significance in showing an association of this type of cracking with greater joint and crack openings and the probability of increased infiltration of foreign material in the openings. For the present, the evidence is not considered to be sufficient to draw any conclusions in this regard.

Spall at Joints and Cracks

Detailed measurements of spall at joints and cracks were made in 1956, four years after construction. In the field survey the distances that the spalls extended along the joints and the spalled widths were measured. In tabulating the data, the spalled areas were placed in four severity classes depending on the width of spall, as follows:

Class I - Up to and including $\frac{5}{8}$ in. width.
Class II - Over $\frac{5}{8}$ in. width up to and including $1\frac{3}{4}$ in. width.
Class III - Over $1\frac{3}{4}$ in. width up to and including 3 in. width.
Class IV - Over 3 in. width.

Data on the extent and severity of spall found in 1956 are presented in Table 11 where the information is tabulated on a per-1,000-lineal-feet-of-pavement basis for comparison. It will be seen from the table that the only significant spall is located at the unedged metal-plate joints and at the transverse cracks that have formed in this plain concrete pavements without joints.

Photographs showing typical conditions at joints and cracks in March 1954 at the end of two winters following construction are presented in Figure 5.

TABLE 11

SERVICE CONDITION OF TRANSVERSE JOINTS AND CRACKS

Test Section	Distinctive Design Features	(1956 Survey) Lineal Feet of Spall at Joints and Cracks Per 1000 Lineal Feet of Pavement					(1957 Survey) Percent of Original Seal Ruptured, Raveled or Missing		
		Class I	Class II	Class III	Class IV	Total	Ruptured in Place	Raveled or Missing	Total
X A	10-in. reinforced concrete; unedged metal-plate joints at 100 ft	11	40	30	23	104	0	100	100
X B	10-in. reinforced concrete; edged metal-plate joints at 100 ft	6	2	1	1	10	22	78	100
1A	10-in. plain conrete; no joints	24	17	4	7	52	-	-	-
1B	10-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	Trace	1	3	1	5	86	14	100
2A	9-in. plain concrete; no joints	12	25	11	4	52	-	-	-
2B	9-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	1	3	2	6	12	79	21	100
3	10-in. reinforced concrete; sawed joints at 100 ft	Trace	3	2	2	7	75	25	100
4	10-in. reinforced concrete; dummy-groove joints at 100 ft (abraded faces)	1	1	1	2	5	68	32	100
5	10-in. reinforced concrete; dummy-groove joints at 100 ft	1	1	1	0	3	60	40	100
6	10-in. reinforced concrete; sawed joints at 100 ft	0	Trace	0	0	Trace	87	13	100
7	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	Trace	5	3	1	9	95	5	100
7 1/	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	Trace	1	1	Trace	2	100	0	100

1/ Hot-poured rubber-asphalt seal

Note 1. The various classes of spall are defined in the text.

Note 2. A cold-applied rubber-asphalt sealing compound used on all joints except on part of Section 7 where a hot-poured rubber-asphalt sealing compound was used.

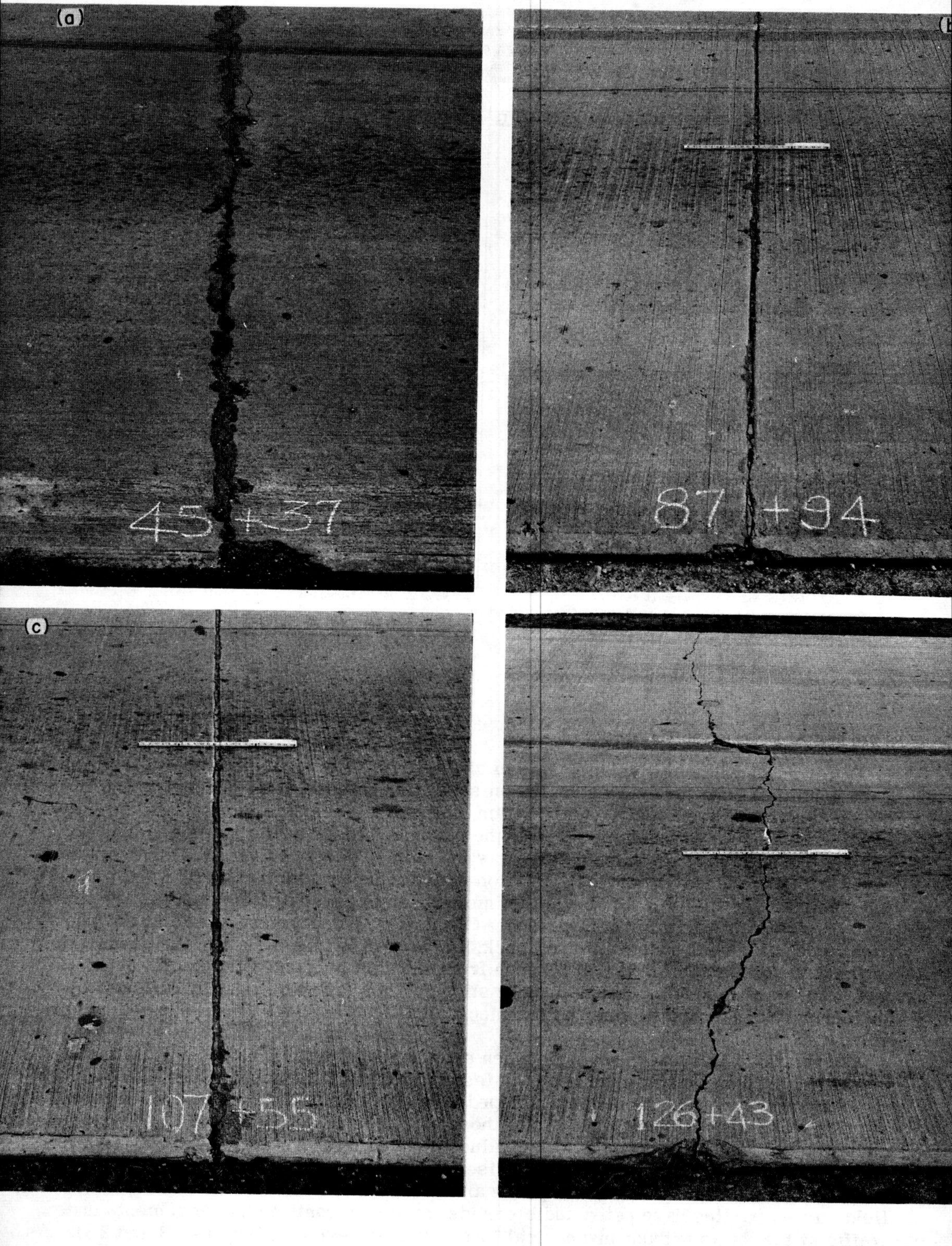
Faulting at Joints and Cracks

Differential vertical displacements of abutting slabs at joints and cracks (faulting) have been measured in the wheelpaths on several occasions in the manner and with the device described previously. Information on the displacements is also available from readings made on the brass plugs near the pavement edges at the joints and cracks.

The most recent survey of faulting was made in 1959, seven years after the pavement had been placed in service. Data on slab displacements that were noted in 1959 are summarized in Table 12 where the information is presented on a per-1,000-lineal-feet-of-pavement basis for comparison. Of the two measurements made in the wheelpaths at each lane at each joint and crack, the larger has been used in the preparation of the data for the table. It will be seen from the data presented that faults at joints and cracks are frequent, though usually slight. Thus far there is little difference in the severity of faulting whether or not mechanical load transfer devices have been provided.

Correlations have been made between extensometer determinations of the average displacement per joint or crack of each test section, and the average test-section faulting per joint or crack, the latter developed from faulting measurements at corresponding times and approximate locations to those of extensometer derivations. Maximum deviation per second between average values derived from the two sources was 0.02 in., with several sections showing precise agreement in values.

Although faulting at transverse joints and cracks was found nowhere to be significant, field survey parties have noted audible evidence of differential slab movements under traffic at the joints without mechanical load-transfer devices (Sections 1B and 2B). As heavier axle loads pass, there are distinct thumping and grinding sounds, indicating that independent movement rather than hinge-like action may be occurring between adjoining slabs.


(a)
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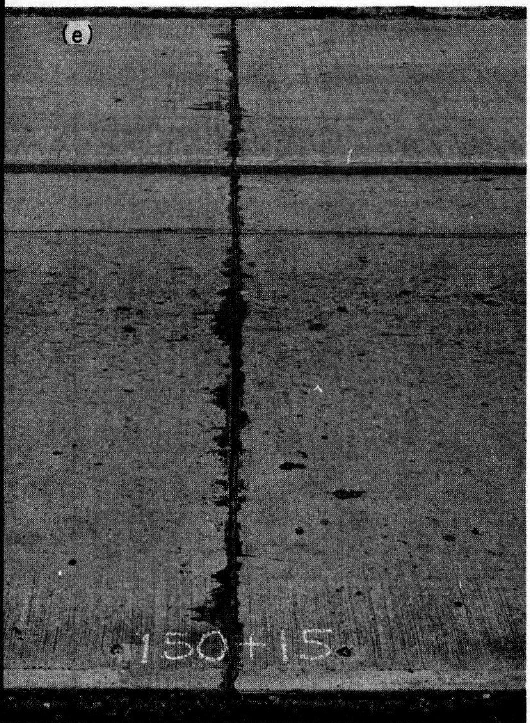

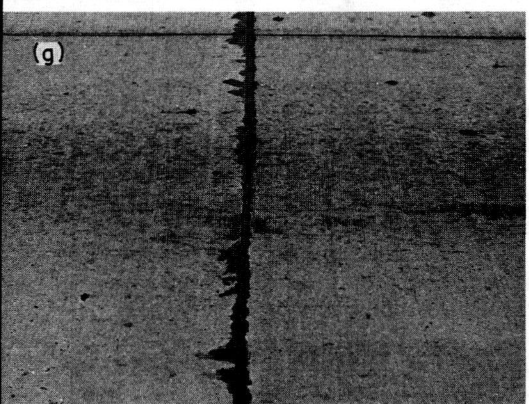

Figure 5. Typical joint and crack conditions two years after construction (March 1954), showing: (a) unedged metal-plate joint, 100-ft spacing, cold-applied sealing compound; (b) edged metal-plate joint, 100-ft spacing, cold-applied sealing compound; (c) dummy-groove joint, 20-ft spacing, cold-applied sealing compound; and (d) crack in nonjointed pavement, no seal applied. Figure 5 (Continued), showing: (e) sawed joint, ½-in. groove, 100-ft spacing, cold-applied spacing compound; (f) dummy-groove joint, 100-ft spacing, cold-applied sealing compound; and (g) dummy-groove joint, 100-ft spacing, hot-poured sealing compound.

TABLE 12
FAULTS AT JOINTS AND CRACKS AND ROAD ROUGHNESS INDEX

Test Section	Distinctive Design Features	(1959 Survey) Joints and Cracks Faulted Per 1000 Lineal Feet of Pavement — Travel Lane: 1/8 In. or More	1/4 In. or More	1/2 In. or More	Passing Lane: 1/8 In. or More	1/4 In. or More	1/2 In. or More	(1959 Survey) Road Roughness Index 1/ — Travel Lane	Passing Lane
		(number)			(number)			(inches per mile)	
XA	10-in. reinforced concrete; unedged metal-plate joints at 100 ft	2	0	0	1	0	0	-	-
XB	10-in. reinforced concrete; edged metal-plate joints at 100 ft	3	0	0	1	0	0	86	83
1A	10-in. plain concrete; no joints	6	1	0	1	0	0	90	82
1B	10-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	9	1	0	2	0	0	115	105
2A	9-in. plain concrete; no joints	7	1	0	2	0	0	85	82
2B	9-in. plain concrete; dummy-groove joints at 20 ft (no dowels)	28	2	0	2	1	0	108	91
3	10-in. reinforced concrete; sawed joints at 100 ft	1	0	0	1	0	0	82	84
4	10-in. reinforced concrete; dummy-groove joints at 100 ft (abraded faces)	1	0	0	1	0	0	79	85
5	10-in. reinforced concrete; dummy-groove joints at 100 ft	2	0	0	1	0	0	75	80
6	10-in. reinforced concrete; sawed joints at 100 ft	1	1	0	1	0	0	70	76
7	10-in. reinforced concrete; edged metal-plate joints at 100 ft (abraded faces)	6	0	0	3	0	0	77	89

1/ Determined with Bureau of Public Roads' type road-smoothness indicator

Surface Smoothness

The Illinois road-smoothness indicator did not become available for use on this project until 1957. The as-constructed road-roughness indexes for the test pavements are therefore not available. Smoothness determinations were made with the device in 1957, and again in 1959. Recorded values of the road-roughness index made in 1959 did not vary appreciably from those made in 1957. Those of 1959 are given in Table 12 where faulting data are also presented. Roughness indexes for the test sections constructed without joints and for those constructed with joints at 100-ft intervals range between 70 and 90 in. per mile. Roughness indexes for the pavements with dummy-groove joints at 20-ft intervals are somewhat higher and lie within the range of 91 to 115 in. per mile.

No consistent relationship between the measured depth at faults at joints and cracks and road-roughness index values is observable up to the present (Table 12).

Condition of Joint Seals

Rupture of the joint seals began the first winter following construction. Both cold-applied and hot-poured seals showed signs of rupture. Rupture was least in the sections of 20-ft joint spacing where only the cold-applied seal was used. This was probably the result of lesser movement at these more closely spaced joints.

The effects, if any, of abrading the joint faces are considered not to be distinguishable. As mentioned previously, the attempts to abrade the joint faces with the device at hand did not result in a uniform removal of the mortar film. Actually, no separation of a mortar film from the joint faces through adherence of the sealing material was found anywhere on this particular project, regardless of whether or not abrasion of the joint faces was attempted.

Summary data concerning the condition of the seals in the winter of 1957 following five years of service are presented in Table 11. It will be noted that the seal at all joints was found to be totally ruptured. Raveling of the seal was found to vary with the section but in most instances no distinct pattern is discernible. The hot-poured material, which was placed in joints with definite grooves or wells, was found to be raveled, whereas the cold-applied material placed under similar circumstances showed raveling in varying amounts. Cold-applied material placed on the unedged joints without grooves was totally raveled. Hot-poured material was not used in the unedged joints.

Reference is again made to Figure 5 where photographs of typical joint and crack conditions two years after construction are shown.

Information relative to measured horizontal movements of slab ends at joints and cracks is presented in the following section.

Joint and Crack Openings

As explained previously, brass reference plugs have been installed in pairs at joints and cracks for use in making precise measurements of vertical and horizontal slab movements. Most of the plugs at the joints were installed in the plastic concrete. All plugs at the cracks and a few at the joints were installed by drilling holes in the hardened concrete and seating the plugs in a mortar mixture. Initial extensometer readings of the distance between the plugs of each pair and of the difference in elevation between the plugs of each pair were made as soon as the plugs became firmly sealed. In the case of the plugs placed in the plastic concrete, readings were made at frequent, though irregular, intervals at each joint until it became apparent that the joint had opened. Readings were scheduled on a seasonal basis thereafter.

Analysis of changes in plug spacings has revealed two characteristics horizontal movements taking place at the joints. The first of the two movements, very slight but measurable, took place before the slab faces at the joints became visibly separated. Following these early movements, the long-term opening and closing cyclical movements became operative.

Early Slab Movements. –The frequent though irregular plug-spacing measurements that were made in the early hours and days following construction showed slight but measurable expansive and contractive movements to be taking place at the transverse joints prior to visible separation of the slabs. Plugs spaced about 24 in. apart frequently showed decreases from the original spacing in the order of 0.003 in., and increases in the order of 0.02 to 0.03 in. before visible separation occurred. Refinements adopted in planning and executing the research on this project were not adequate for following these movements with precision.

Slab separations occurred at all transverse joints during the first few days following construction. The observations in this respect are not considered to be of particular significance except possibly in connection with the different methods of curing used on the two sawed-joint sections.

As noted previously, behavior comparisons between wetted-burlap and impermeable-paper curing methods allied with the use of sawed joints were planned. Conclusive results of this research (curing methods in respect to sawed joints) failed to materialize because no random cracking occurred prior to or during sawing in either of two sawed-joint test sections. However, the time period between construction and slab separation was longer by several days on the wet-burlap-cured section than on any of the sections cured with impermeable paper.

No important differences in climatic conditions occurred during the period that the burlap-cured pavement was under observation, so it seems possible that the lack of early separation may have been due to the difference in curing. However, the evidence furnished by the present study is not sufficient to be considered conclusive.

Long-Term Slab Movements. –Long-term slab movements at joints and cracks have occurred in both vertical and horizontal directions. Averages of the vertical displacements measured at joints of the individual test sections during the first five years of pavement life are shown in Figure 6. It will be noted that average displacements are very slight thus far, although there is an apparent tendency toward an increase with age. The forward slabs in the direction of traffic are, on the average, becoming lower than the abutting slabs. No significant differences in the behavior of the individual test sections are apparent.

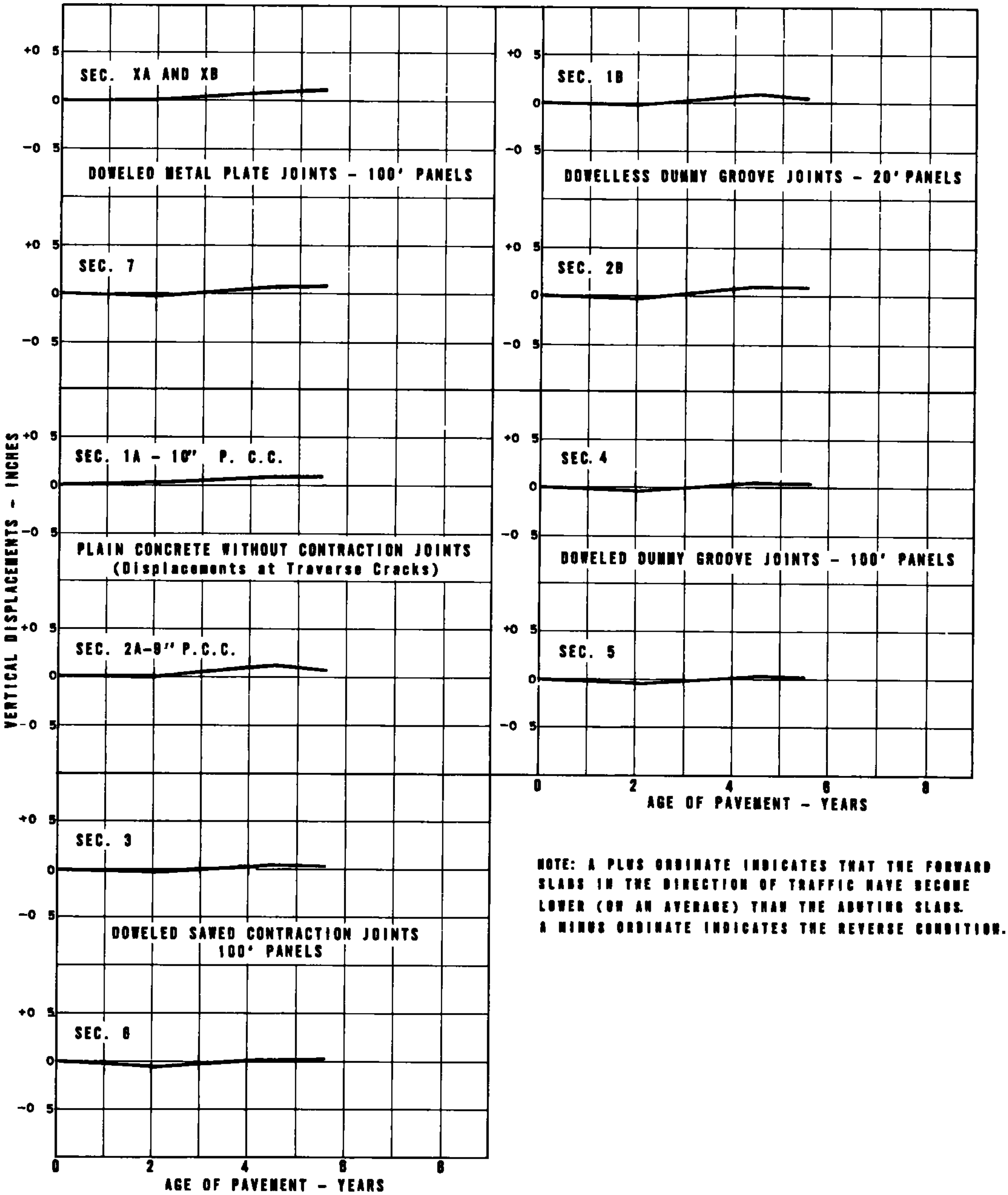

Figure 6. Vertical displacements of abutting ends of pavement slabs.

A conventional analysis of horizontal slab movement at joints and cracks is depicted in Figure 7. Although gaps in the field data reduce the value of the analysis, trends that were to be anticipated from the results of similar studies elsewhere are apparent. Opening and closure movements are least for the closely spaced joints (Sections 1B and 2B having 20-ft panels). Joints and cracks of all sections show a tendency to not return to complete closure. Joints spaced at 100-ft intervals maintain greater openings during hot weather than do the joints spaced at 20-ft intervals.

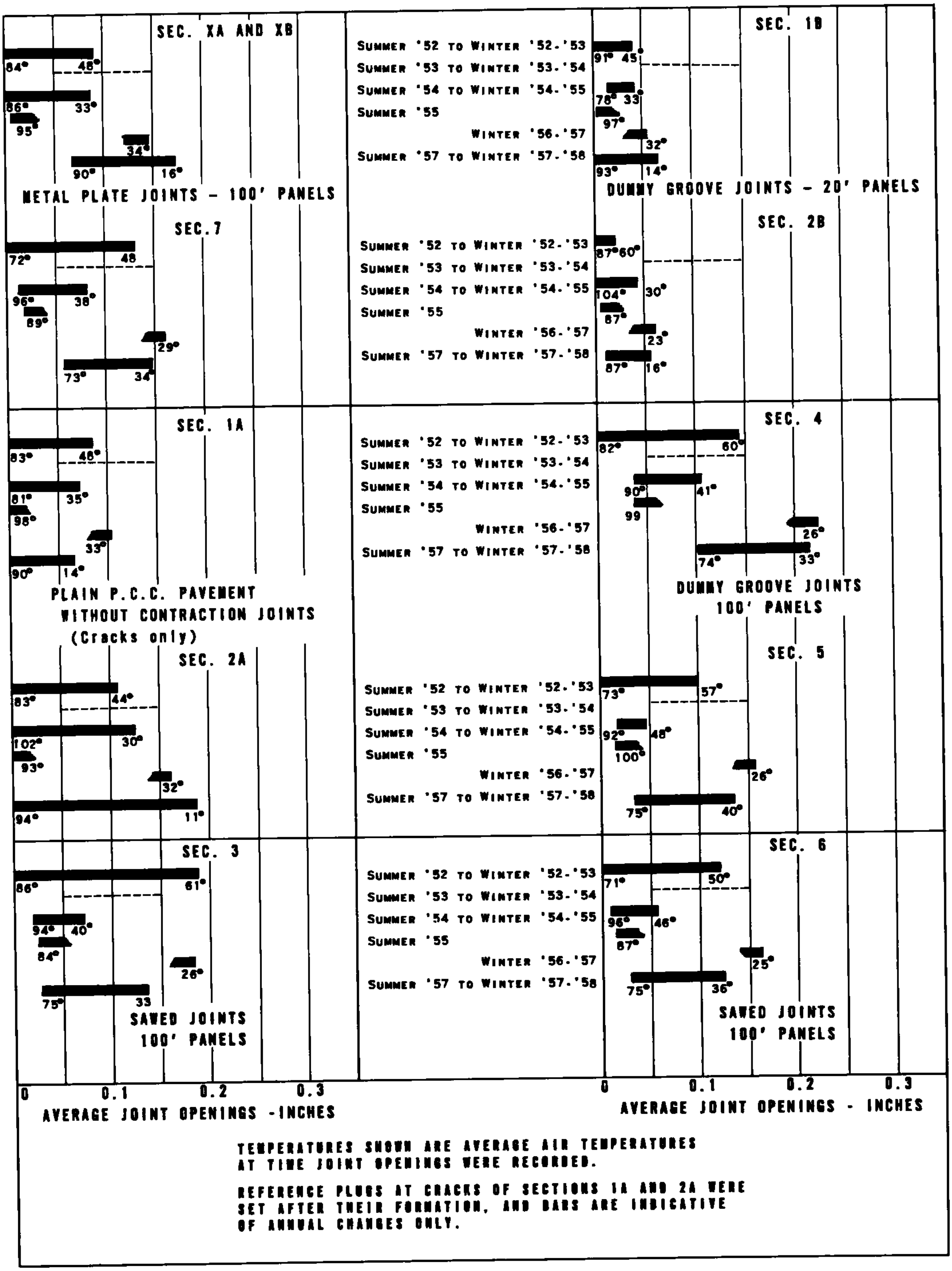

Figure 7. Annual and progressive changes in openings of transverse joints and cracks.

INTERIM FINDINGS

After seven years of service all of the experimental pavement is in acceptable condition, and such defects as have been noted are of a relatively minor nature. Nevertheless, some of the defects that have occurred have been concentrated on certain of the test sections and do offer a means of comparing the various experimental design features with respect to resistance to these defects. Pavement defects that have been noted thus far have been limited to transverse cracking, the formation of infiltration cracks, spall at joints and cracks, differential displacement of slabs at joints and cracks, and ineffectiveness of the various joints in the retention of seal. The riding quality of the various test sections as indicated by a road-smoothness indicator has also been determined.

Defects

Interim findings with respect to the various defects that have occurred are:

Transverse Cracking. —Transverse cracking is confined almost wholly to the plain unjointed test sections. A question arises as to whether the plain unjointed pavements should be compared adversely with the other pavements because of this concentration of transverse cracking in them. Inasmuch as the average uncracked slab length for the unjointed pavement is within the same range as the average uncracked slab length for the pavement with joints at 100-ft intervals, and is much greater than that for the pavement with joints at 20-ft intervals, it is considered that the unjointed pavements should not be looked on in an adverse light at the present time solely because of this cracking.

Infiltration Cracks. —Infiltration cracks are few in number and show no real pattern of occurrence with respect to the experimental designs. There is slight indication that they are more prevalent where the longer joint spacings were used.

Spall. —Spall, as would be expected, has occurred where edging was omitted at the full-depth metal-plate joints. Such spall is widespread but not of great depth or width. Spall is also much in evidence at the cracks in the unjointed pavements. This spall is also widespread and in a few instances quite severe with respect to depth and width. Very little spall has occurred elsewhere.

Faults. —Slab faulting is present at many of the joints and cracks. Differential displacements are slight, and not related to any particular joint design including the joints without mechanical load transfer devices. However, because the joints without load transfer devices are spaced at 20-ft intervals as compared with the 100-ft intervals of the joints with load transfer devices, the number of faulted slabs per unit length of pavement is greater for the sections containing these joints.

Riding Quality. —Measurements of riding quality made in 1957 and 1959 with a standard road-smoothness indicator showed the pavements jointed at 20-ft intervals to be inferior to the unjointed pavements and to the pavements with joints at 100-ft intervals. Other experimental features did not produce significant differences in riding quality as determined with the road-smoothness indicator. The roughness-index values of 70 to 90 in. per mile that were determined for the jointless pavements and for the pavements with joints at 100-ft intervals correspond with those determined with the same device for the majority of standard-design pavements with joints at 100-ft intervals for which roughness-index values have been determined in Illinois. The roughness values of 91 to 115 in. per mile for the pavements with joints at 20-ft intervals correspond with values determined for the rougher standard-design pavements in the state.

Shoulder Holes. —Pumping has not been noted on the test project. Shoulder holes without evidence of ejected material have been noted at the pavement edges near joints and cracks in a few instances, but their occurrence has been so infrequent and so random in location that little significance has been attached to them.

Condition of Seal. —Of two rubber-asphalt sealing materials which were used, one a hot-poured material and the other a cold-applied material, the hot-poured material showed somewhat better adhesion to the concrete and greater resistance to cracking and removal. Neither material was ever completely effective in cold weather under the conditions of this particular project.

The abrading of joint faces of hand-edged joints to remove surface mortar of the type that occasionally has been found pulled away from the main body of concrete by sealing materials caused some undesirable spall and did not produce uniformly abraded faces with the methods and equipment which were used. There was no significant evidence to indicate that the sealing material adhered to the concrete better at the abraded joints.

Experimental Features

In consideration of the distribution of the various defects that have occurred, interim findings with respect to the experimental features are:

Pavement Design.—The best service is being rendered currently by Illinois standard steel-reinforced pavement with joints at 100-ft intervals and load-transfer devices at the joints. The jointless pavements without steel reinforcement, and the nonreinforced pavements with joints at 20-ft intervals and no load transfer devices are rendering satisfactory service, though not in such a high degree as the standard pavement. The jointless pavements of 9-in and 10-in. thickness are of approximately equal behavior after seven years of service, as are also the 9-in. and 10-in. thick pavements with joints at 20-ft intervals.

Contraction Joints.—Little difference has been noted thus far in the behavior of the impressed joints with hand-tooled edges, the standard metal-plate joints with hand-tooled edges, and the sawed joints. Non-edged metal-plate joints show considerable slight spall and poor retention of seal. Joints with and without load-transfer devices show little difference in behavior thus far; however, the differences in the joint spacing of these two designs must be considered as a probable factor when making this comparison.

Sealing Compounds.—Under the conditions of this particular project, the hot-poured rubber-asphalt sealing compound has shown somewhat better adhesence to the concrete and less tendency toward raveling and rupturing than has the cold-applied rubber-asphalt seal. Neither type of seal has remained unbroken during cold weather. At seven years neither material appeared to be functioning and all joints were resealed.

Abrasion of Joint Faces.—The device used for abrading joint faces for the purpose of removing any weak mortar film that might have accumulated did not provide continuous abrasion. There was no significant evidence to indicate that the abrasion process as used on this project was truly effective.

Curing.—In the study designed to determine the influence of impermeable-paper curing and wetted-burlap curing on premature cracking where sawed joints are used, no cracks developed on either test section. However, some observations of slab separation at the transverse joints indicated that there was less movement during the first few days after construction where the wetted-burlap cure was used.