Report on Experimental Project in Michigan

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● DURING the construction of the Michigan Test Road in 1940 and subsequent to it, many observational and special studies have been made in addition to carrying out the program of seasonal physical measurements which was set up in the original outline. The test road has been in existence now for 10 years and during this period there has been collected and analyzed a considerable amount of data. It is the purpose of this report to present as concisely as possible the significant trends in slab and joint behavior as well as other interesting disclosures which are believed to be of sufficient interest in relation to present design and construction of concrete pavements.

Since the Michigan Test has been fully described in a Department bulletin entitled, "The Michigan Test Road" published in July, 1942, in the Highway Research Board's Research Report No. 3 B published in November 1945 and just recently in a new publication, August 1950, by the Department titled "Michigan Test Road -- Design Project," repetition of basic information has been purposely avoided in this report except where necessary for a better understanding of the results.

The important design features included in the project for study are joint spacing, joint design, pavement cross section, steel reinforcement, uniform thickness versus balanced cross section, stress curing, and relation of pavement cross section to subgrade supporting values. In addition such construction features as mechanical spreading of concrete, mechanical tamping of forms, and joint sealing compounds were included for observational study.

In order to evaluate the design features previously mentioned under controlled conditions it was necessary to divide the project into 12 test areas. The test areas designated as Series 1 to 12, are described in Table 1 entitled "Summary of Test Areas." The table includes important information pertinent to each test area. To facilitate the study of a particular design feature each series has been further subdivided into divisions and sections designated by letters and numerals, respectively.

The most outstanding contributions derived from the test road studies included in the Design Project have been: (1) the use of bituminous-rubber joint seal materials; (2) mechanical form tamping; (3) the change to long slabs with heavier steel and no intermediate plane of weakness joints; (4) the use of groove type contraction joint construction; (5) the use of heavier and more closely spaced dowel bars for load transfer in transverse joints; and (6) the elimination of expansion joints except at designated locations and during fall construction.

This report contains miscellaneous project information pertaining to soil, traffic and climatic conditions, and includes also a discussion of the general behavior of joints, slab movement and several incidental studies associated with the project.

Miscellaneous Project Information

During the construction of the pavement surface and subsequent to it, certain important factual data have been procured which are directly or indirectly associated with the general behavior of the pavement slabs. Such information includes general soil conditions, physical properties of the concrete, traffic conditions and climatological data.

GENERAL SOIL CONDITIONS

The subgrade materials are composed, primarily, of well-drained sandy or gravelly soils with the exception of two areas, Stations 88+00 to 129+00 and Stations 170+00 to 225+06 where it was necessary to construct a 12-inch sand subbase over existing subgrade material. The granular subbase and subgrade materials, in general, fall into Bureau of Public Roads Soil Classification A-3, whereas the soil material lying within the above stations meets the Bureau classification for A-4 and A-6 soils. The physical properties of subgrade soil of four representative locations where a subbase was not required are given in Table 2.

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*Deceased

TABLE 1

SUMMARY OF TEST AREAS

Test Area Designation		Number of Sections in Division	Length of Division in feet	Pavement Thickness, inches	Reinforcement lb./100 sq. ft.	Joint Spacing in feet			Load Transfer Type			Filler and Seal	Special Factors Under Study
Series	Division					Expansion	Contraction	Dummy	Expansion (1)	Contraction (2)	Dummy (3)	Expansion Joint (4)	
S		1	600	9-7-9	60	120	60	30	DB-1	DB	R	1	
1	A	3	360	9-7-9	60	120	60	30	DB-1	DB	R	1	Joint Spacing
	B	3	720	9-7-9	60	240	60	30	DB-1	DB	R	1	Joint Design
	C	3	1440	9-7-9	60	480	60	30	TE	DB	R	1	Reinforcement
	D	2	1800	9-7-9	60	900	60	30	DB-1	DB	R	1	Expansion Space
	E	1	1800	9-7-9	60	1800	60	30	DB-1	DB	R	1	
	F	1	2700	9-7-9	60	2700	60	30	DB-1	DB	R	1	
2	F	1	2700	9-7-9	37	2700	30	15	DB-1	DB	R	1	Joint Spacing
	E	1	1800	9-7-9	37	1800	30	15	DB-1	DB	R	1	Joint Design
	D	2	1800	9-7-9	37	900	30	15	DB-1	DB	R	1	Reinforcement
	C	3	1440	9-7-9	37	480	30	15	TE	DB	R	1	Expansion Space
	B	3	720	9-7-9	37	240	30	15	DB-1	DB	R	1	
	A	3	360	9-7-9	37	120	30	15	DB-1	DB	R	1	
3	A	3	360	9-7-9	None	120	20	None	DB-1	DB	None	1	Joint Spacing
	B	3	720	9-7-9	None	240	20	None	DB-1	DB	None	1	Reinforcement
	C	3	1440	9-7-9	None	480	20	None	DB-1	DB	None	1	Contraction joints with and
	D	2	1800	9-7-9	None	900	20	None	DB-1	DB	None	1	without load transfer devices
	E	1	1800	9-7-9	None	1800	20	None	DB-1	None	None	1	Expansion Space
	F	1	2700	9-7-9	None	2700	20	None	DB-1	DB	None	1	
4	F	1	2700	9-7-9	None	2700	10	None	DB-1	DB	None	2	Joint Spacing
	E	1	1800	9-7-9	None	1800	10	None	DB-1	None	None	2	Reinforcement
	D	2	1800	9-7-9	None	900	10	None	DB-1	DB	None	2	Contraction joints with and
	C	3	1440	9-7-9	None	480	10	None	DB-1	DB	None	2	without load transfer devices
	B	3	720	9-7-9	None	240	10	None	DB-1	DB	None	2	Expansion Space
	A	3	360	9-7-9	None	120	10	None	DB-1	DB	None	2	
5	A	3	360	9-7-9	37	120	30	None	DB-1	1B	None	3	Contraction Joint Design
	B	3	360	9-7-9	37	120	30	None	DB-1	2A	None	3	Reinforcement
	C	3	360	9-7-9	37	120	30	None	DB-1	2B	None	3	
	D	3	360	9-7-9	37	120	30	None	DB-1	3	None	3	
	E	3	360	9-7-9	37	120	30	None	DB-1	3	None	3	
	F	3	360	9-7-9	37	120	30	None	DB-1	4	None	3	
	G	3	360	9-7-9	37	120	30	None	DB-1	4	None	3	
6	A	5	600	8	None	120	30	None	CB-1	CB	None	2	Cross Section
	B	5	600	8	None	120	20	None	CB-1	CB	None	2	Joint Design
	C	2	600	8	None	300	15	None	CB-1	CB	None	2	Reinforcement
	D	2	600	8	None	300	10	None	CB-1	CB	None	2	
7	A	5	600	8-6-8	60	120	60	30	DB-1	DB	R	2	Cross Section
	B	5	600	8-6-8	37	120	30	15	DB-1	DB	R	2	Reinforcement
	C	5	600	8-6-8	None	120	20	None	DB-1	DB	None	2	
	D	5	600	8-6-8	None	120	10	None	DB-1	DB	None	2	
8	A	3	360	7	None	120	30	None	CB-1	CB	None	2	Cross Section
	B	7	840	7	None	120	20	None	CB-1	CB	None	2	Reinforcement
	C	2	600	7	None	300	15	None	CB-1	CB	None	2	Joint Design
	D	2	600	7	None	300	10	None	CB-1	CB	None	2	
9	TS	1	180	9-7-9	None	180	30	None	TB	DB	None	4	Stress Curing
	A	1	1800	9-7-9	None	100	None	None	TB	None	None	4	Joint Design
	TS	1	90	9-7-9	None	180	30	None	TB	DB	None	4	
	TS	1	90	9-7-9	None	180	30	None	DB-1	5	None	4	
10	A-1	9	1080	9-7-9	None	120	20	None	DB-1	DB	None	5	Contraction joints with and
	A-2	9	1080	9-7-9	None	120	15	None	DB-1	DB	None	5	without load transfer devices
	B-1	9	1080	9-7-9	None	120	20	None	A	None	None	2	
	B-2	9	1080	9-7-9	None	120	15	None	A	None	None	2	
11	A	1	90	9-7-9	60	90	None	None	TA	None	None	6	Continuous slab construction
	B	1	120	9-7-9	60	120	None	None	TA	None	None	6	with reinforcement
	C	1	362	9-7-9	60	362	None	None	TA	None	None	6	
	D	1	600	9-7-9	60	600	None	None	TA	None	None	6	
12	A	1	90	9-7-9	None	90	None	None	TA	None	None	6	Continuous slab construction
	B	1	120	9-7-9	None	120	None	None	TA	None	None	6	without reinforcement
	C	1	360	9-7-9	None	360	None	None	TA	None	None	6	
	D	1	242	9-7-9	None	242	None	None	TA	None	None	6	
	E	1	600	9-7-9	None	600	None	None	TA	None	None	6	

(1) EXPANSION JOINT CONSTRUCTION:
Type DB-1 — ¾" x 15" dowel bar expansion joint assembly. Dowels at 15" spacing.
Type TE — Thickened edge 1¼" x 18" corner dowel bar expansion joint assembly. Dowels 9" from slab edge.
Type CB-1 — Unthickened edge, 1¼" x 18" corner dowel bar expansion joint assembly. Dowels 9" from slab edge.
Type TB — Translode base expansion joint assembly.
Type TA — Translode angle unit expansion joint assembly.
Type A — No load transfer feature.

(2) CONTRACTION JOINT CONSTRUCTION:
Type DB — ¾" x 15" dowels at 15" spacing, premolded filler.
Type 1B — ¾" x 15" dowels at 15" spacing, groove and poured seal.
Type 2A — ¾" x 15" dowels at 15" spacing, premolded filler, metal parting strip at bottom.
Type 2B — ¾" x 15" dowels at 15" spacing, groove and poured seal, metal parting strip at bottom.
Type 3 — ¾" x 15" dowels at 15" spacing, groove and poured seal, full depth metal divider plate.
Type 4 — Continuous plate dowel assembly.
Type 5 — Keylode contraction joint assembly.
Type CB — 1¼" x 18" dowels at corners, 9" from slab edge, premolded filler.
Type 6 — Aggregate interlock. No dowels.

(3) DUMMY PLANE OF WEAKNESS JOINTS:
R — Aggregate interlock, steel mesh reinforcement continuous through joint.

(4) EXPANSION JOINT, FILLER AND SEAL:
Type 1 — Premolded fiber filler with asphalt-latex seal.
Type 2 — Premolded fiber filler with asphalt-vultex seal.
Type 3 — Air chamber with top, bottom and sides sealed with asphalt-latex compound.
Type 4 — Air chamber with premolded rubber seal at top, bottom and sides, asphalt-latex seal in bottom.
Type 5 — Premolded fiber filler with thermoplastic seal.
Type 6 — Premolded fiber filler with SOA seal.

TABLE 2

PHYSICAL PROPERTIES OF SOIL AT MOISTURE CELL STATIONS

	Station 722+10	Station 851+80	Station 1055+75	Station 61+05
Gravel, % retained, No. 18 sieve	15	5	6	26
Sand, % retained, No. 270 sieve	84	91	90	72
Silt, % retained, 0.005 mm.	1	3	3	2
Clay, % retained, 0.001 mm.	0	1	1	0
Liquid limit	19	19	20	18
Plasticity index	Non-Plastic	Non-Plastic	Non-Plastic	Non-Plastic
Specific gravity	2.62	2.61	2.65	2.63
Shrinkage limit, %	No Shrinkage	No Shrinkage	No Shrinkage	No Shrinkage
Loss on ignition, %	0.67	0.80	1.39	0.61
Organic content, %	0.62	0.64	1.36	0.45
Capillary rise, inches	7	12.0	10	10.5
Field moisture equivalent, %	19	18	20	17
Moisture, bottom in. of rise, %	24.9	23.9	23.0	20.2
Moisture, top in. of rise, %	6.7	4.7	5.4	5.0
Coefficient of permeability, ft. per day	26	52	38	40
Weight on samples, psi	0.6	0.6	0.6	0.6
Voids, %	30.8	32.0	32.0	30.8

TABLE 3

PHYSICAL PROPERTIES OF CONCRETE

	Compressive Strength psi		Flexural Strength psi		Modulus of Elasticity 10^6 pounds per square inch	
	12-in. cylinders	6-in. dia. cores	6- by 8- by 24- in. beams			
	28 days	21 months	7 days	28 days	at 500 psi	at 1000 psi
Low	2880	3780	439	518	6.35	6.05
High	5360	7185	718	849	7.22	6.59
Average	5203	5643	376	697	6.89	6.30

Coefficient of Thermal Expansion 0.0000053

Consistency - Slump Cone Method - 1 to 3.5 in. - Average . . . . 2.03 inches

Weight per Cubic Foot . . . . 153 pounds

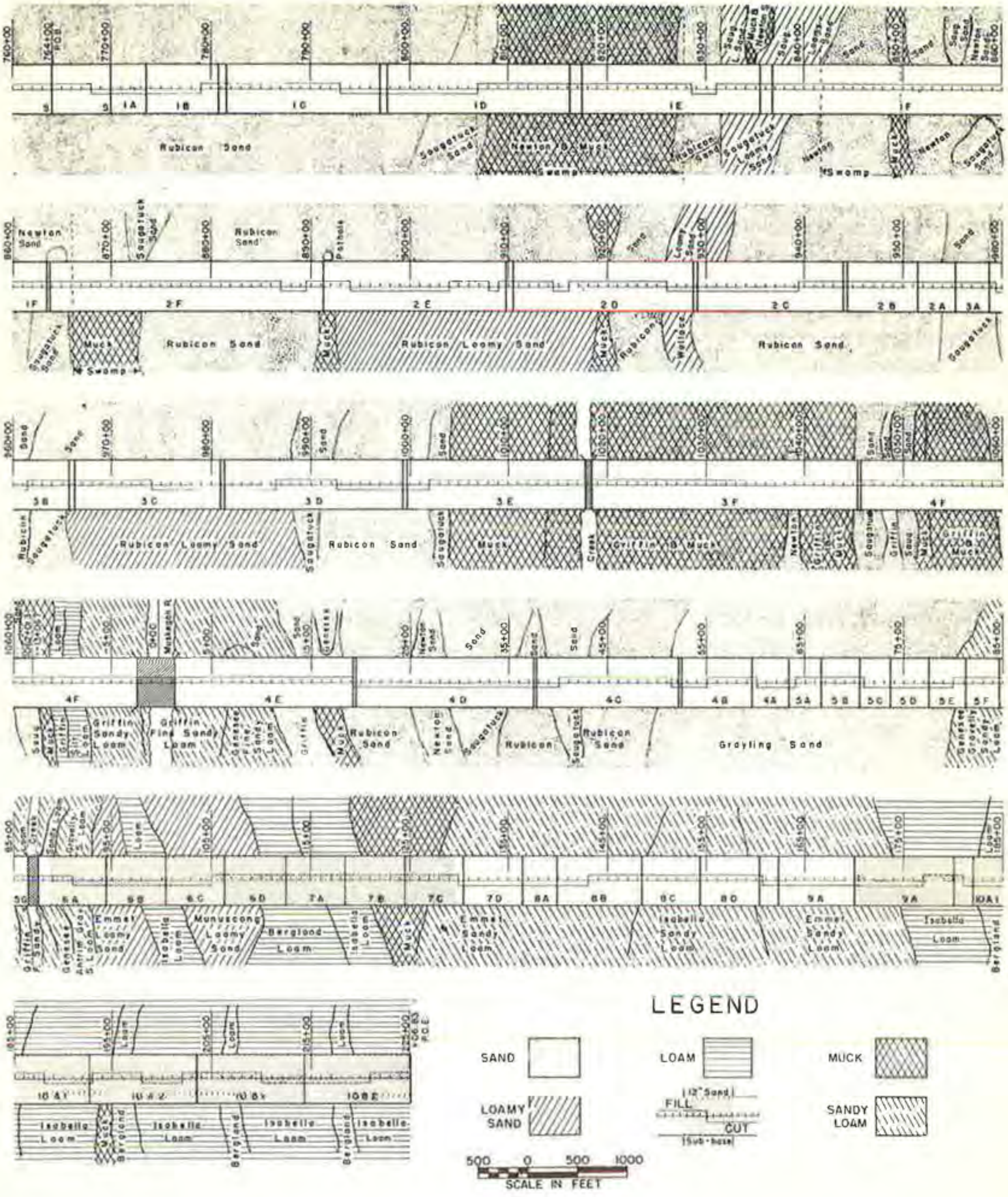

750+00
754+00 P.O.B.
770+00
780+00
790+00
800+00
830+00
850+00
860+00
1A
1B
1C
1D
1E
1F
Rubicon Sand
Saugatuck Sand
Newton / B Muck
Swamp
Newton
Muck
860+00
870+00
880+00
890+00
900+00
910+00
930+00
940+00
950+00
960+00
2F
2E
2D
2C
2B
2A
3A
Newton Sand
Rubicon Sand
Pothole
Rubicon Loamy Sand
Wallace
960+00
970+00
980+00
990+00
1000+00
1050+00
1060+00
3B
3C
3D
3E
3F
4F
Rubicon
Saugatuck
Griffin / B Muck
Creek
Muskegon R.
4E
4D
4C
4B
4A
5A
5B
5C
5D
5E
5F
Griffin Sandy Loam
Griffin Fine Sandy Loam
Genesee Fine Sandy Loam
Rubicon Sand
Newton Sand
Grayling Sand
85+00
105+00
115+00
125+00
135+00
145+00
155+00
165+00
175+00
185+00
6A
6B
6C
6D
7A
7B
7C
7D
8A
8B
8C
8D
9A
10A1
Genesee
Emmet Loamy Sand
Isabella Loam
Munuscong Loamy Sand
Bergland Loam
Emmet Sandy Loam
Isabella Sandy Loam
Isabella Loam
Bergland
205+00
225+00
226+83 P.O.E.
10A1
10A2
10B1
10B2
LEGEND
SAND
LOAM
MUCK
LOAMY SAND
FILL
(12" Sand)
CUT
(Sub-base)
SANDY LOAM
500
0
500
1000
SCALE IN FEET

Figure 1. Soil types and earthwork operations.

The density of the soil at a point 9 inches below the bottom of the slab at the time of placing of concrete slab ranged from 103 to 113 pcf. Moisture content of soil at time of measurement varied from 4.2 to 7.6 percent of dry weight of soil.

Subgrade performance has been satisfactory throughout the project with the exception of several frost heave areas which have developed in Series 6 and 9. The effect of the frost heave on slab performance will be discussed later under the physical condition of the respective series.

The extent and relative location of soil types and earthwork operations for the Design Project are illustrated in Figure 1.

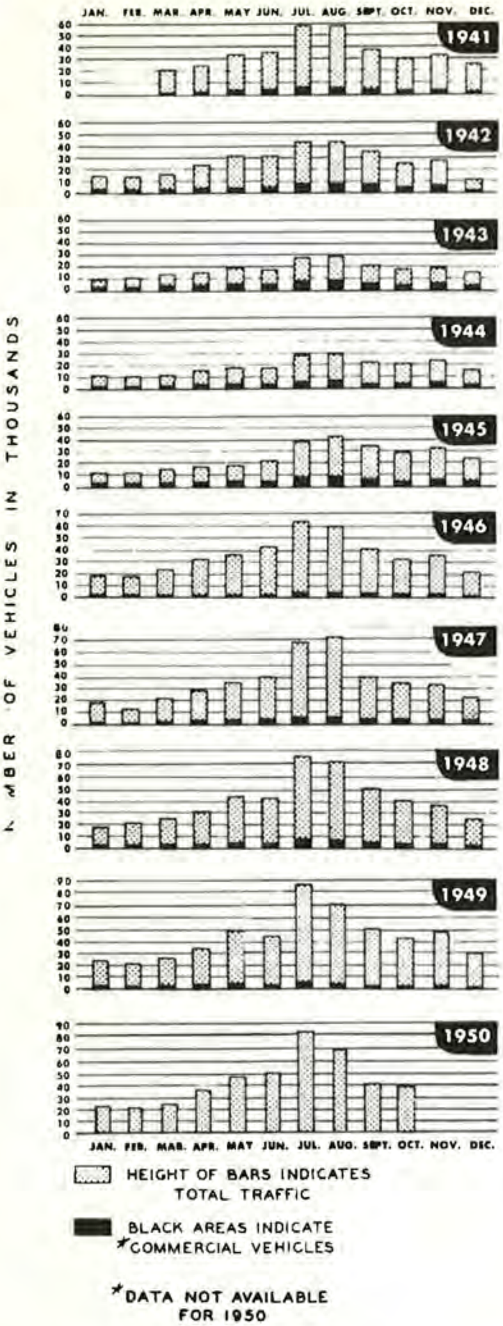


Figure 2. Monthly traffic record.

PHYSICAL PROPERTIES OF CONCRETE

Certain physical properties of the concrete such as weight per cubic foot, consistency, compressive and flexural strength, modulus of elasticity, and coefficient of thermal expansion are given in Table 3.

TRAFFIC CHARACTERISTICS

Automatic recording equipment was installed on the test road to obtain a continuous daily record of traffic flow. From 1941 to 1949, traffic classification surveys were made quarterly — April, July, October, and December — covering a 6-hour period per day for five days. The 6-hour periods were rotated around the clock in order that data representative of a 24-hour day for the different seasons could be obtained for each year. Starting with 1950, the above traffic classification procedure was changed to a continuous 24-hour period once each month. Similar surveys on other highways indicate that such a procedure gives better results.

During these surveys the axle loads, axle spacings, and frequency of various types of commercial vehicles are recorded. Wheel loads are obtained by means of portable loadometers from which axle loads may be obtained.

Normal monthly traffic flow on the test road is presented graphically in Figure 2. Values representing the percentages of different types of vehicles traveling the test road based on an average annual day for the years 1941 to 1949 inclusive are given in Table 4. Annual average wheel load distribution values by direction of travel are presented in Table 5. Figure 3 presents a graphic comparison of axle load frequencies on the test road with that of a normal heavy primary route.

CLIMATOLOGICAL DATA

The graph in Figure 4 shows that the average temperature in winter is approximately 25 deg. F while in summer it is 70 deg. F, making an average temperature difference of about 45 deg. F. It may be observed also that daily temperature fluctuations in winter are about 16 deg. F less than those occurring during the summer.

Total yearly precipitation data for 1941 to 1948 inclusive is given in Table 6. The data indicates an annual average rainfall of 29. 5 inches for the test road.

Concrete Pavement Performance

The major pavement design studies under consideration are the spacing and design

TABLE 4

CLASSIFICATION OF ANNUAL AVERAGE DAILY TRAFFIC

Classification	1941 No.	1941 %	1942 No.	1942 %	1943 No.	1943 %	1944 No.	1944 %	1945 No.	1945 %	1946 No.	1946 %	1947 No.	1947 %	1948 No.	1948 %	1949 No.	1949 %
Total traffic	1590	100.0	829	100.0	578	100.0	733	100.0	803	100.0	1204	100.0	1176	100.0	1361	100.0	1472	100.0
Passenger cars	1437	90.4	668	80.6	394	68.2	583	79.5	666	82.9	1117	92.8	1066	90.7	1238	91.0	1368	92.9
Total commercial	153	9.6	161	19.4	184	31.8	150	20.5	137	17.1	87	7.2	110	9.3	123	9.0	104	7.1
Light	43	2.7	44	5.3	14	2.4	32	4.4	16	2.0	4	0.3	5	0.4	29	2.1	6	0.4
Medium	26	1.6	37	4.4	56	9.7	51	7.0	27	3.4	30	2.5	28	2.4	25	1.8	32	2.2
Heavy	43	2.7	12	1.5	13	2.2	5	0.7	5	0.6	7	0.6	18	1.5	5	0.4	5	0.4
Trailer combinations	41	2.6	68	8.2	101	17.5	62	8.4	89	11.1	46	3.8	59	5.0	64	4.7	61	4.1

TABLE 5

ANNUAL AVERAGE WHEEL LOAD DISTRIBUTION

	1945				1946				1947				1948				1949			
	SE Bound (South Lane)		NW Bound (North Lane)		SE Bound (South Lane)		NW Bound (North Lane)		SE Bound (South Lane)		NW Bound (North Lane)		SE Bound (South Lane)		NW Bound (North Lane)		SE Bound (South Lane)		NW Bound (North Lane)	
Wheel Load	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Under 4000	302	62.27	293	59.07	391	67.76	287	63.21	387	54.28	501	65.24	318	52.22	428	68.70	403	70.0	402	54.1
4000- 4499	19	3.92	13	2.62	19	3.29	12	2.64	23	3.23	32	4.17	33	5.42	24	3.85	22	3.8	22	3.0
4500- 4999	19	3.92	6	1.21	8	1.39	11	2.42	19	2.66	17	2.21	18	2.96	22	3.53	15	2.6	17	2.3
5000- 5499	20	4.12	11	2.22	18	3.12	7	1.54	22	3.09	24	3.13	29	4.76	22	3.53	8	1.4	22	3.0
5500- 5999	13	2.68	11	2.22	18	3.12	19	4.19	20	2.81	23	2.99	17	2.79	14	2.25	21	3.6	20	2.7
6000- 6499	21	4.33	15	3.03	23	3.99	19	4.19	22	3.09	18	2.34	19	3.12	17	2.73	17	3.0	41	5.5
6500- 6999	19	3.92	20	4.03	18	3.12	18	3.97	24	3.37	19	2.47	19	3.12	20	3.21	17	3.0	36	4.8
7000- 7499	17	3.50	22	4.44	18	3.12	12	2.64	26	3.65	23	2.99	20	3.28	27	4.34	16	2.8	29	3.9
7500- 7999	19	3.92	39	7.86	20	3.47	19	4.19	40	5.61	27	3.52	29	4.76	17	2.73	22	3.8	31	4.2
8000- 8499	11	2.27	39	7.86	11	1.90	22	4.85	38	5.32	26	3.39	26	4.27	15	2.41	11	1.9	50	6.7
8500- 8999	9	1.85	17	3.43	12	2.08	14	3.08	38	5.32	24	3.13	31	5.09	5	0.80	13	2.2	38	5.1
9000- 9499	10	2.06	8	1.61	10	1.73	9	1.98	34	4.77	21	2.73	19	3.12	9	1.44	3	0.5	19	2.6
9500- 9999	6	1.24	2	0.40	9	1.56	5	1.10	14	1.96	8	1.04	15	2.46	2	0.32	5	0.9	11	1.5
10000-10499					2	.35	0		4	0.56	2	0.26	8	1.31	1	0.16			4	0.5
10500-10999									1	0.14	3	0.39	5	0.83			2	0.3		
11000-11499									1	0.14			1	0.16					1	0.1
11500-11999													2	0.33			1	0.2		
Total axles	485	100.00	496	100.00	577	100.00	454	100.00	713	100.00	768	100.00	609	100.00	623	100.00	576	100.00	743	100.00
Total vehicles	170		178		210		165		594		270		200		214		207		259	
Ratio axles to vehicles	2.85		2.79		2.65		2.75		1.20		2.84		3.04		2.91		2.78		2.87	

Sampling consists of taking one 6-hour sample per day for five consecutive days at four periods in each year – January, April, July, October. The time of taking the 6-hour samples is changed for each period to give a 24-hour sample per year.

TABLE 6

ANNUAL PRECIPITATION RECORD

Year	Precipitation in inches
1941	31.03
1942	28.91
1943	29.48
1944	21.66
1945	37.39
1946	30.37
1947	29.33
1948	27.86
Average	29.50

of transverse joints, pavement cross section, and steel reinforcement.

The evaluation of the several features included in these major design studies will be based upon the behavior of the respective concrete slabs or pavement sections under service conditions, taking into account joint width movement, structural performance, physical irregularities, and roughness.

PAVEMENT PERFORMANCE IN RELATION TO JOINT SPACING

Although joint spacing is considered throughout the entire Design Project, it has received special emphasis in Series 1, 2, 3, and 4. In these four series expansion joints have been spaced to give sections of 120, 240, 480, 900, 1800, and 2700 foot lengths, and contraction joints have been spaced at 10, 20, 30, and 60 foot intervals. Dummy, or so-called warping joints are included in the sections containing 60 and 30 foot contraction joint spacing. Contraction joints are plane of weakness joints with or without slip dowels or other types of load transfer devices, whereas dummy joints are constructed in the same manner except that they do not contain load transfer devices and the pavement reinforcement is continuous through the joint.

Initial measurements of joint width and slab position were made immediately upon completion of each series in the summer and fall of 1940, and the readings have been used as a reference in determining subsequent displacements. Seasonal and daily readings were taken as nearly as possible at the same time of day during all periods of observation. Since the time required to make all measurements for the entire project covers a period of three to four weeks, fluctuations in climatic conditions from day

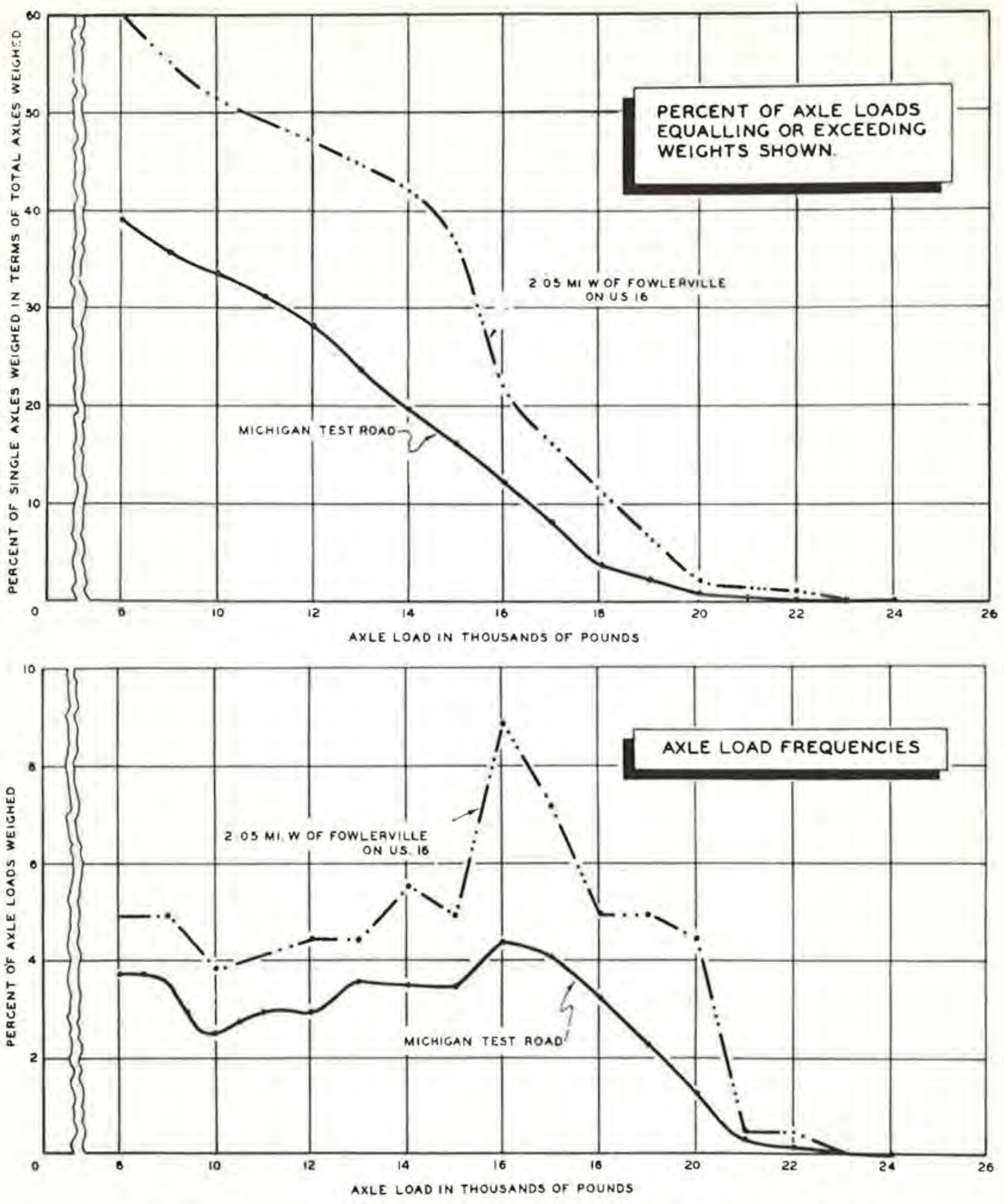


Figure 3. Comparison of axle loads on test road with primary Route US 16.

to day will naturally influence the seasonal measurements between series to a certain extent. Joint width readings are undoubtedly affected to some extent by the curling of the slabs also. The effect of these day to day changes in slab conditions during the observation period has not been considered in the presentation and interpretation of the data in this report.

The period of taking joint width measurements was dependent to a large extent upon weather conditions. In general, the spring readings were taken during the latter part of April and the first part of May, summer measurements include those taken in July and August, fall readings were usually taken in October and November, and winter

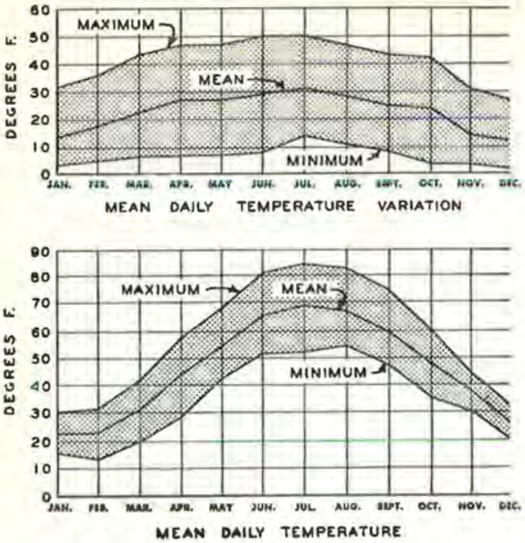


Figure 4. Temperature record.

readings any time from January to March. Winter readings were taken when temperatures were seasonable and the pavement surface was sufficiently free of snow and ice to permit measurements.

The joint width movements of the different test sections have been reduced to average curves which represent the average seasonal movement for all joints under observation in any given section. This has been necessary because of the vast amount of data accumulated over the past ten years. Spring and fall readings were discontinued in 1948.

The results from these joint studies will be discussed under seasonal changes in joint widths, daily changes in joint widths, and pavement movement.

Seasonal Changes in Joint Widths

In presenting the data on seasonal changes in joint widths, each type of joint under study will be discussed separately. The joints given major consideration in this investigation include individual expansion joints, and relief sections composed of two or more 1-inch expansion joints, contraction joints, and dummy or plane of weakness joints.

Expansion Joints. Seasonal changes in expansion joint width for the several sections in Series 1-2-3 and 4, together with their progressive or permanent change, are presented graphically in Figure 5 for the years 1941 to 1950, inclusive. These graphs also show the relationship between change in joint width and length of section between expansion joints. Unless otherwise stated, only those expansion joints separating sections of equal length were considered in plotting the graph. Where relief sections are involved, consisting of two or more expansion joints separated by small slabs of concrete, the individual expansion joint movements were combined algebraically to form a single value representative of one joint or equivalent width.

Figure 5 presents the joint width movement of certain expansion joints in relation to length of section after the joint width readings had been adjusted to an average summer temperature of 75 deg. F and an average winter temperature of 25 deg. F, using coefficients derived from daily movements.

Several significant facts are revealed by the graphs in Figure 5. (1) In most cases the sections contracted sufficiently during the first winter season to cause a slight widening of the expansion joints in excess of the 1-inch width originally provided. (2) Without exception all of the sections experienced their greatest movement during the first year after construction. (3)The annual amplitude of joint width movement diminishes with time. (4) All expansion joints show a progressive, permanent change in joint width resulting in a gradual closing of the joints, to the extent that after 10 years the sections have absorbed approximately 60 to 80 percent of the expansion space provided. There is no doubt that the progress of these residual displacements will diminish rapidly in the future, since the joint filler will eventually reach a stage of compaction sufficient to resist practically all further movement of the slabs adjacent to the joint. (5) As one would expect, the longer sections produced the greatest changes in joint width the first year, although the amplitude of annual joint width movement after the first year is comparable to that of the shorter sections. (6) The amplitude of yearly movement was the least for the sections composed of 10-foot contraction joints and greatest for the sections with 60-foot contraction joints. This phenomenon would indicate that a considerable amount of section movement is absorbed by the greater number of contraction joints existing in a section containing 10-foot contraction joints.

Contraction Joints. The actual changes in contraction joint widths for different sea-

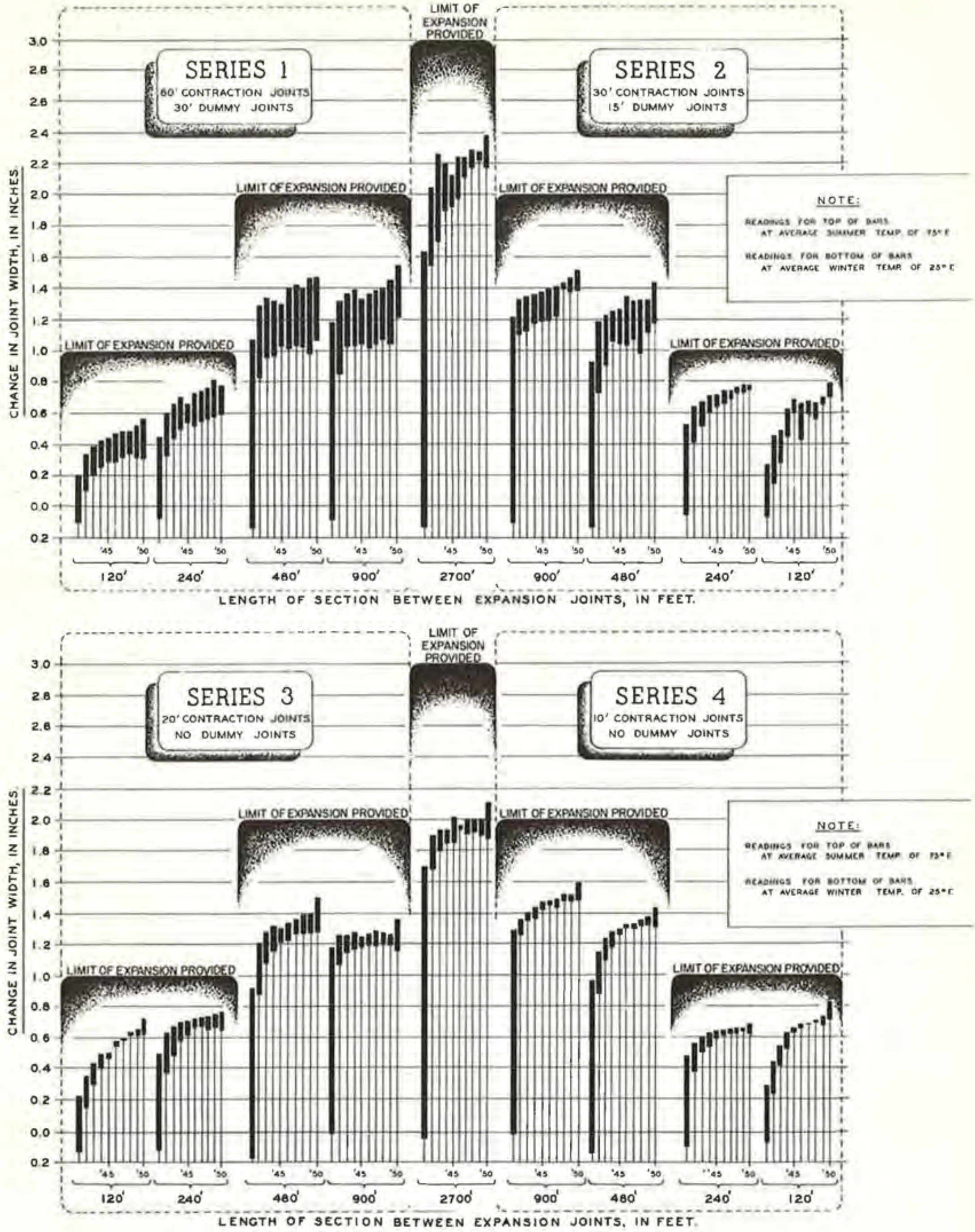
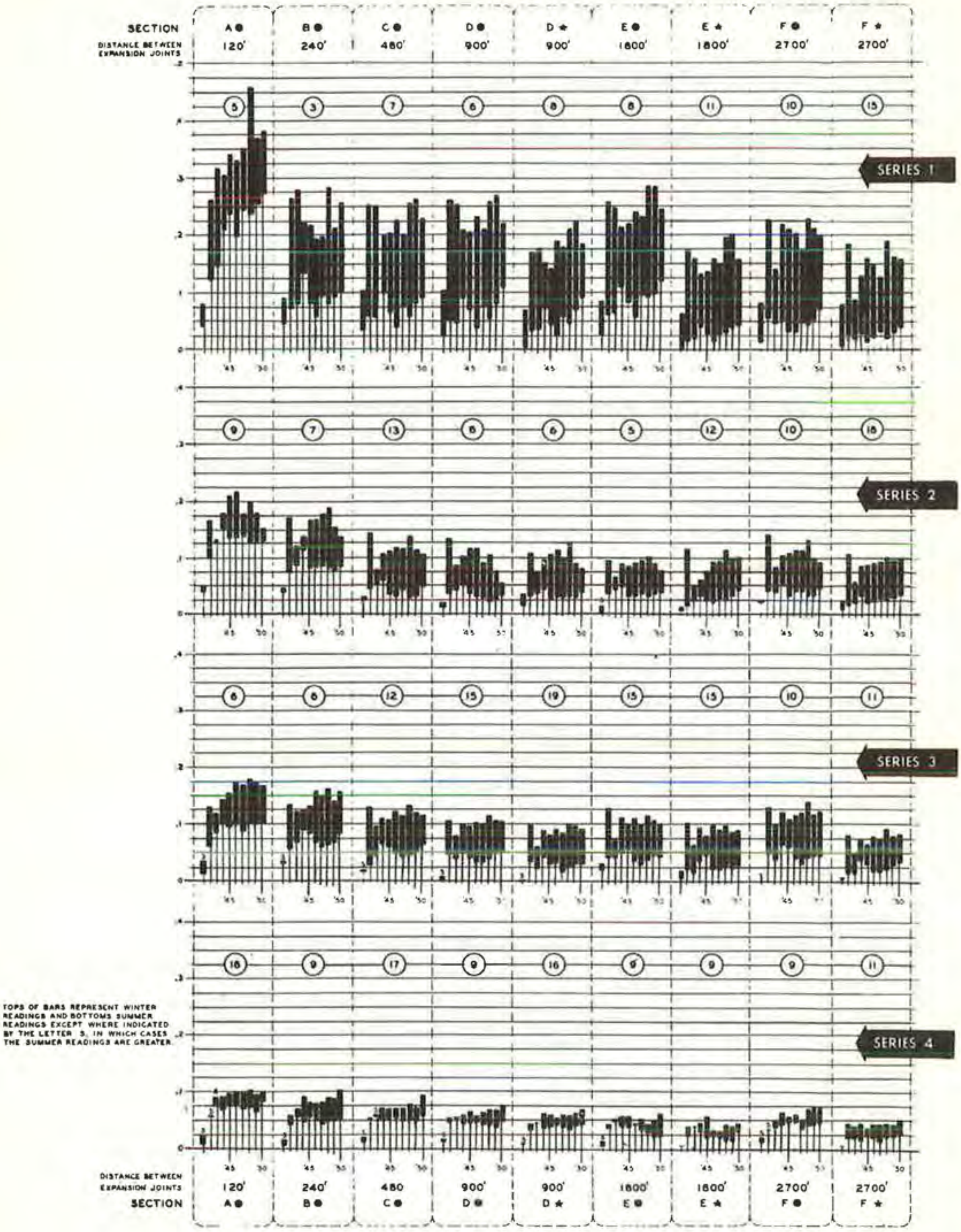


Figure 5. Annual and progressive changes in expansion joint width.

sons of the year and for the variable expansion and contraction joint spacings included in Series 1-2-3 and 4 are shown graphically in Figure 6. The graphs show the average joint width movements for summer and winter seasons.

The relative change in contraction joint width for three particular joint spacings — 60, 20, and 10 feet — are graphically presented in Figure 7. In addition to showing the effect of joint spacing upon joint width changes, these graphs show the residual opening of the joints with time and that the joints closest to the expansion joints open more than the joints near the center portion of the section. This same phenomenon



TOPS OF BARS REPRESENT WINTER READINGS AND BOTTOMS SUMMER READINGS EXCEPT WHERE INDICATED BY THE LETTER S, IN WHICH CASES THE SUMMER READINGS ARE GREATER.

NOTE
NUMBERS IN CIRCLES INDICATE NUMBER OF CONTRACTION JOINTS USED IN AVERAGES.

LEGEND: ★ MIDDLE OF SECTION ● NEAR EXPANSION JOINT

Figure 6. Seasonal changes in contraction joint width.

may be observed on all of the test sections.

The relationship between seasonal contraction joint width movement based on a 10 year average is disclosed by the following data.

	Expansion Joint Spacing in feet					
	120	240	480	900	1800	2700
	in.	in.	in.	in.	in.	in.
Series 1	.125	.140	.153	.136	.146	.126
Series 2	.047	.068	.068	.061	.053	.062
Series 3	.060	.065	.055	.054	.056	.057
Series 4	.020	.031	.020	.017	.017	.019

The following significant facts are disclosed by the graphs in Figures 6 and 7: (1) that under similar conditions of expansion joint spacing the movement of the 60-foot contraction joints is at least four times as great as those spaced at 10 feet; (2) the changes in width of contraction joints vary with the section length, the shorter the section length the greater the contraction joint movement; (3) in the long sections the movement of the contraction joints near the expansion joints is slightly greater than that of the joints near the center of the section; (4) the contraction joints show an annual amplitude of joint width change which apparently decreases with time, the amplitude being greater in the longer slabs and diminishing with decrease in slab length; and (5) with few exceptions, all contraction joints experienced a gradual progressive increase in width during the first five years, and very little increase in residual opening thereafter. The seasonal variation in joint width is still very pronounced, however, under certain design conditions.

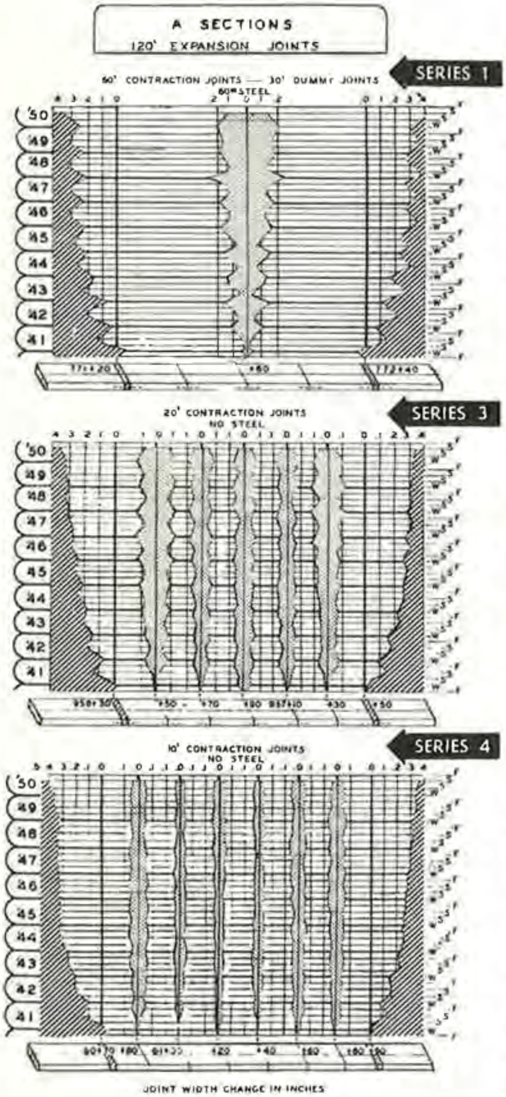
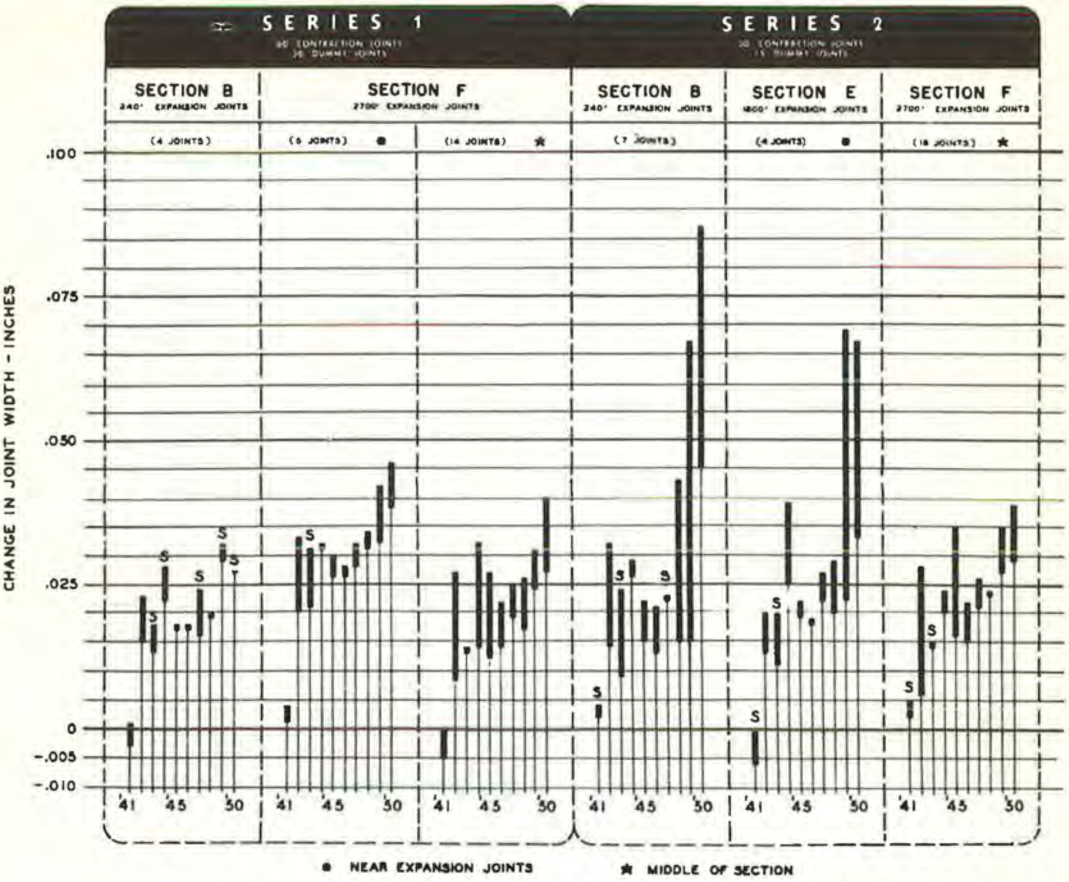


Figure 7. Contraction joint residual movement.

Dummy Joints. In Series 1 and 2, 60-lb. and 37-lb. per 100 square feet mesh reinforcement, respectively, was laid continuously through the dummy joints. Measurements have been taken at several joint locations throughout Series 1 and 2 to study the effect of the reinforcement upon joint behavior. Average seasonal changes in joint width are shown by graphs in Figure 8.

The graphs show that in practically all cases the maximum opening of joints does not exceed 0.05 inches. As in the case of contraction joints, the movement of the dummy joints near the center of the long sections is less than that of joints near the ends. The graphs also indicate that the dummy joints react in the same manner as contraction joints but to a much smaller degree, in that they fluctuate slightly with seasonal changes and seem



TOPS OF BARS REPRESENT WINTER READINGS AND BOTTOMS SUMMER READINGS EXCEPT WHERE INDICATED BY THE LETTER S, IN WHICH CASES THE SUMMER READINGS ARE GREATER

Figure 8. Seasonal changes in dummy joint width.

to acquire a small, gradually increasing residual opening with time. No relation is apparent between joint width change and weight of steel reinforcement. In the eighth and ninth years, however, several dummy joints in Sections B and E of lightly reinforced Series 2 have opened excessively during the winter, indicating a break in the steel at those points.

Daily Changes in Joint Widths

In conjunction with the seasonal joint width measurements certain joints were selected for daily observations. Readings on the same joints were taken early in the morning while the pavement was cool and then in the mid-afternoon when the pavement would be normally at its maximum temperature. The relationships for the daily joint width movements for all series are expressed in comparable terms, such as change in joint width in inches per degree Fahrenheit versus length of section and spacing of joints. Daily readings were discontinued in January 1948.

Expansion Joints. The average daily changes in expansion joint widths by years and seasons are represented by bar graphs in Figure 9. Included in the graphs are measurements from selected joints in all ten sections of the Design Project. In general, the data disclose several significant facts. (1) Daily joint width movement is influenced to a certain extent by the degree of pavement restraint which normally increases with age due to depletion of expansion space and residual volume changes in the concrete. (2) Intermediate contraction joint spacing has a decided effect upon daily joint width

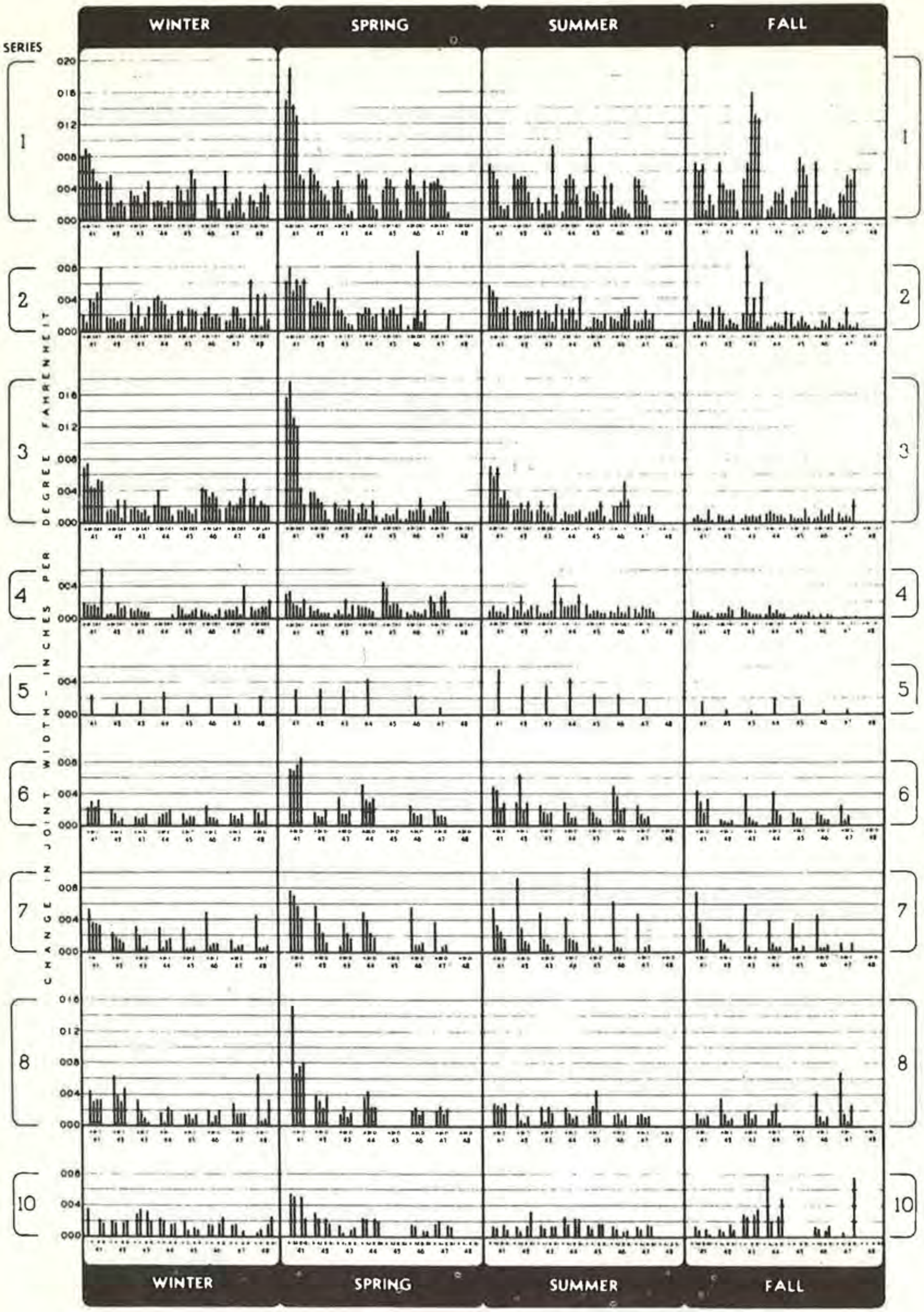
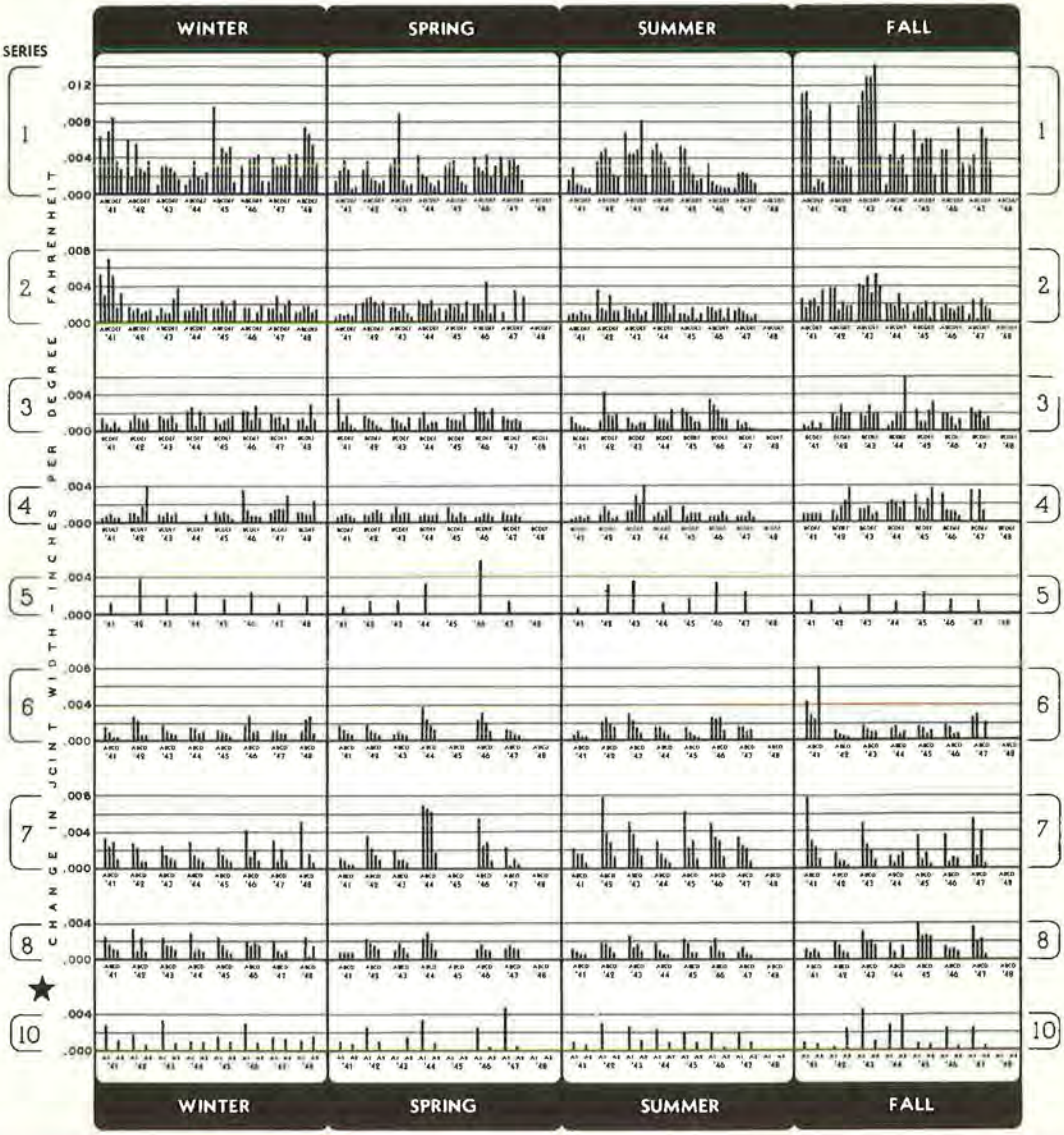


Figure 9. Daily movement of expansion joints based on average of individual joints.

movement as may be observed by comparing graphs of Series 1 with those of Series 4. (3) In general, the movement is greatest during the spring and least in the fall seasons, while summer and winter seasonal readings are about comparable. It is believed that this greater movement in the spring than in the fall may be due to the relatively greater freedom of the slab resulting from winter opening of the joints combined with a wider temperature range induced by the radiant heat of the sun, which is maximum at the summer solstice (June 21). (4) No definite relationship is discernible between daily joint width movement and certain construction features such as weight of reinforcement, cross section, thickness, or joint design.

It is believed that the exceptionally high daily movements for all series in section lengths greater than 240 feet are due to the fact that greater expansion space was provided in those cases. Two 1-inch joints were used for the 480 and 900 foot sections, and three 1-inch joints for all sections 1800 feet and 2700 feet in length.



★ SERIES 9 HAS NO CONTRACTION JOINTS

Figure 10. Daily movement of contraction joints.

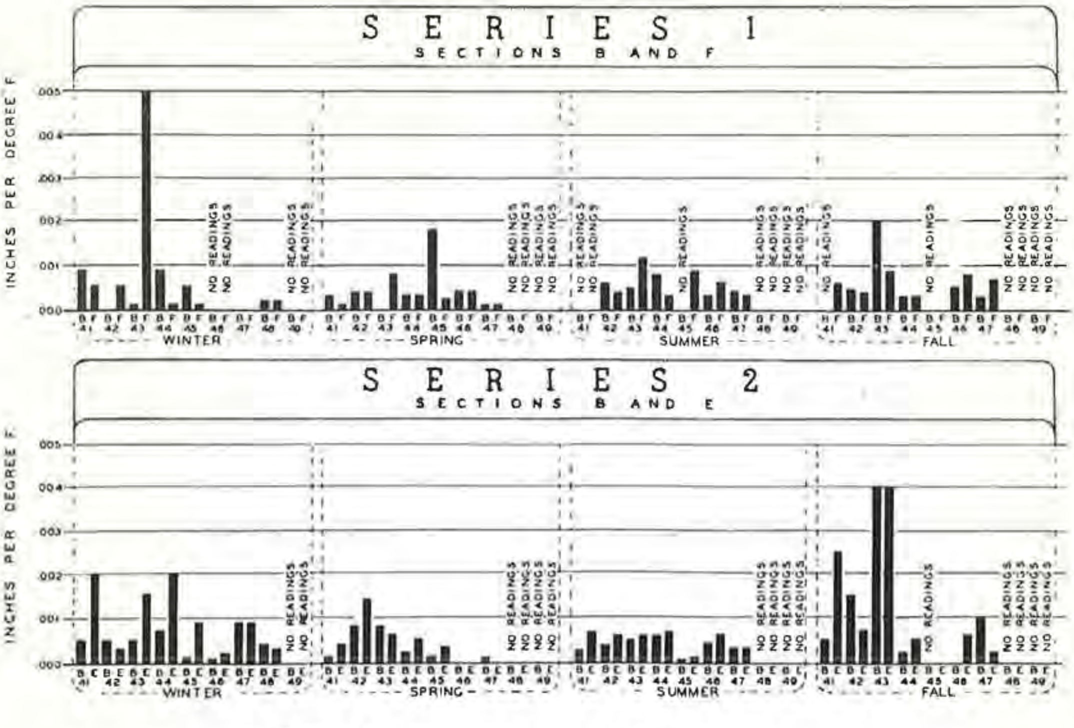


Figure 11. Daily movement of dummy joints.

Contraction Joints. In a similar manner the average daily contraction joint width movements have been presented in Figure 10. The data presented in Figure 10 shows in general that the contraction joints behave in the same manner as the expansion joints. Because of the greater number of joints involved in the case of contraction joints, the relationships between joint width movement and joint spacing are more pronounced.

Dummy Joints. Daily observations have been made on certain dummy joints in Series 1 and 2. Data from these observations are presented graphically in Figure 11. The graphs serve to show that the joints function in the manner of other joints but to a lesser degree, and that the magnitude of the daily movement is in general under 0.001 of an inch per degree F.

Pavement Movement

In certain sections of Series 1, 2, 3, and 4 reference monuments were established to measure the relative movement of different parts of the sections with respect to fixed points in the subgrade. Monuments were placed at the center, quarter points and ends of Sections 1A, 1F, and 4F and at the ends and midpoints of Sections 3A, 4A, 1C, 4C, 1D, 3D, 2F, and 3F. The curves in Figures 12 to 15 inclusive show the relative behavior of the different parts of each section, in respect both to seasonal movement and to the distance of the monument from the center of the section.

The data indicate that for long sections of pavement the greatest movement is at the ends and rapidly diminishes until a point is reached at which practically no longitudinal movement takes place. This is clearly shown by graphs presented in Figure 16. For the two 2700-foot sections – Series 1 and Series 4 – the point of zero longitudinal movement was, in 1941, approximately 700 to 800 feet from the ends of the sections but in 1950 the same point had retreated slightly to 1000 and 1100 feet from the ends. It is also noted in Figure 16 that the two sections have acquired a considerable increase in residual displacement during the 10-year interim. The substantially greater movement of the north end of Section 4F is due to the presence of five 1-inch expansion joints at the relief end instead of the usual three expansion joints because of the abut-

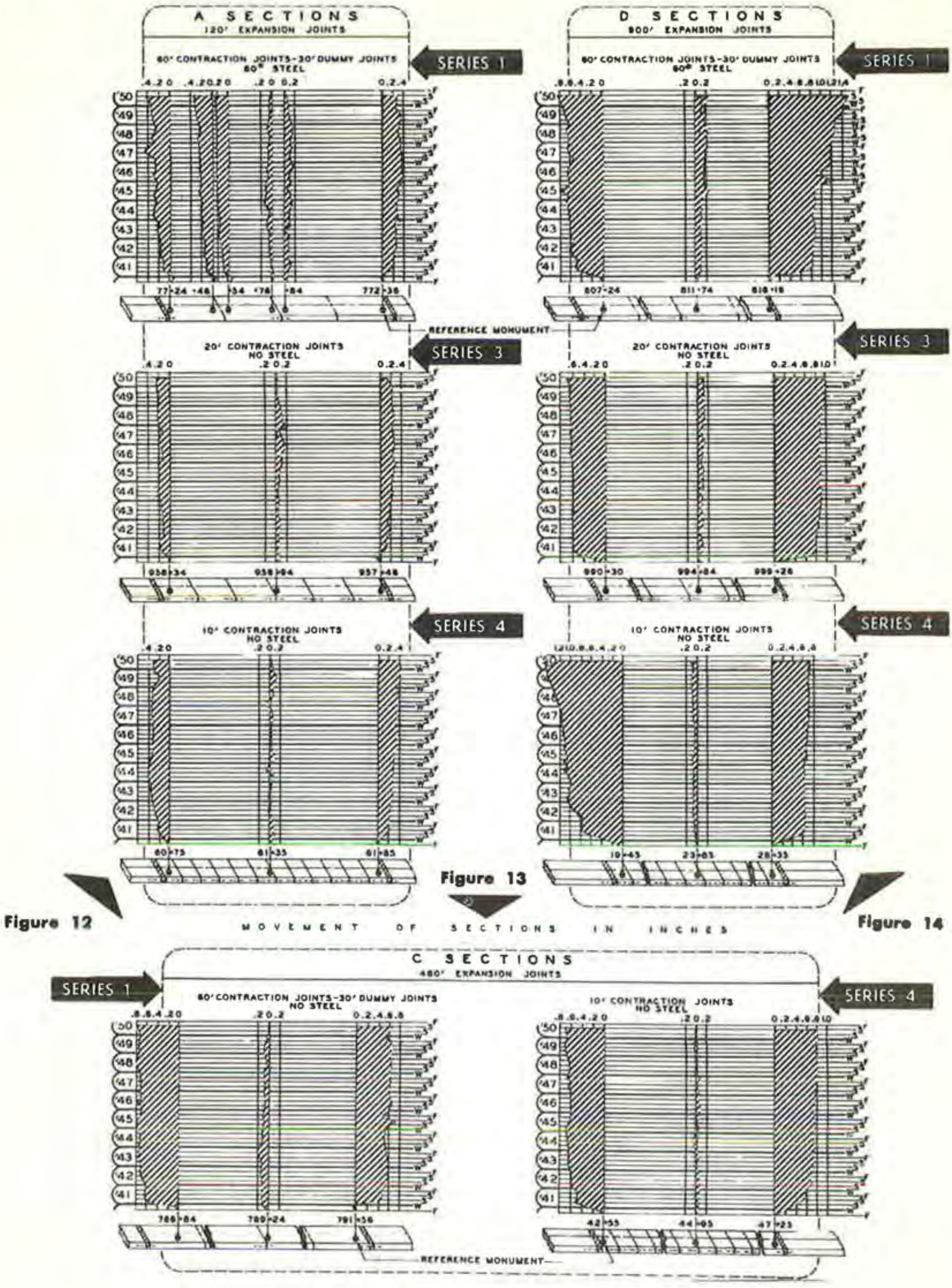


Figure 12

Figure 13

Figure 14

Figures 12, 13 & 14. Seasonal changes in section length.

ting Muskegon River bridge. Thus, there exists in the central part of the 2700-foot sections in Series 1, 2, 3, and 4, portions of pavement more than 500 feet along which at elevated temperatures are under restraint similar to that of a continuous slab without expansion joints. Therefore, in the case of sections whose lengths are less than

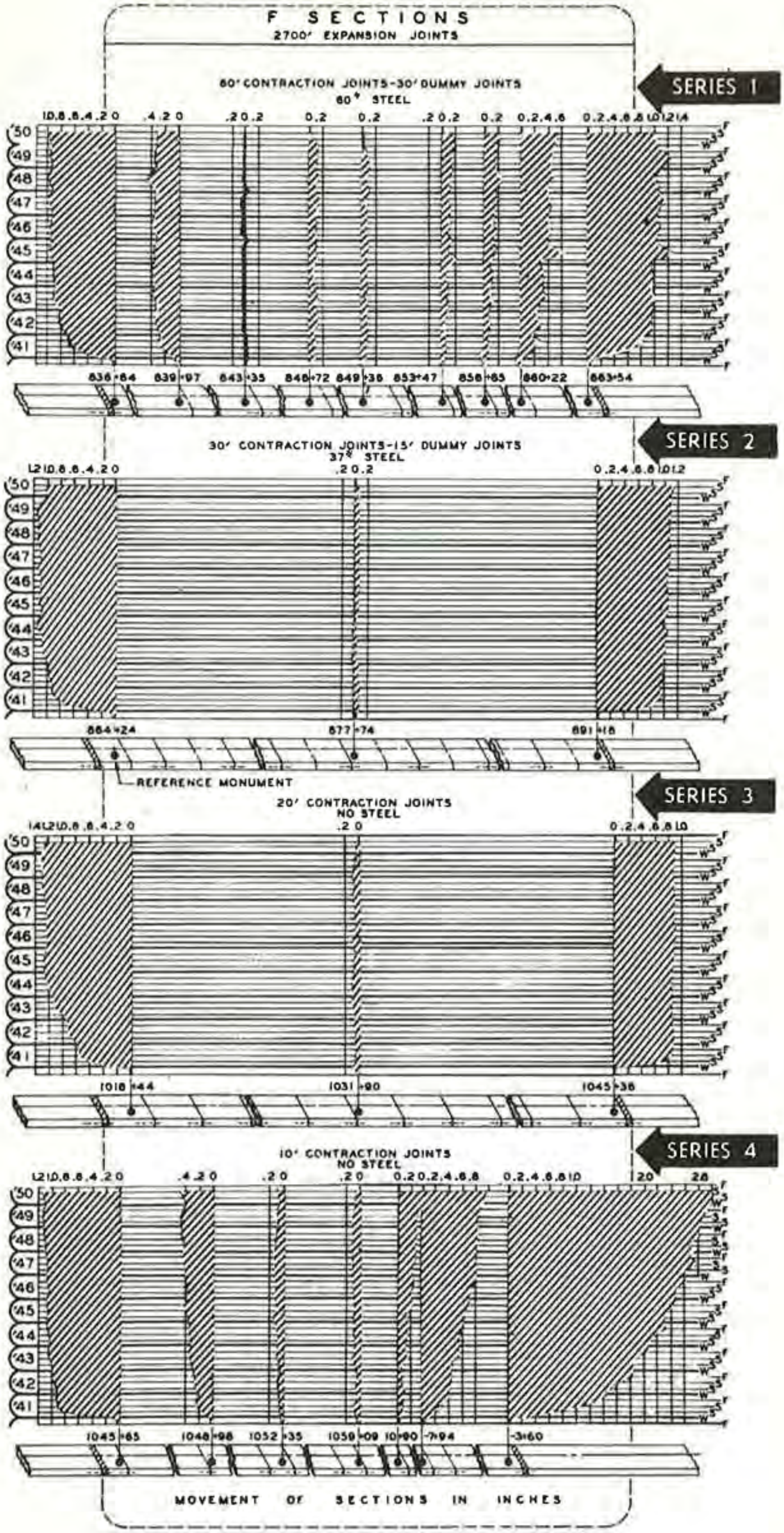


Figure 15. Seasonal changes in section length.

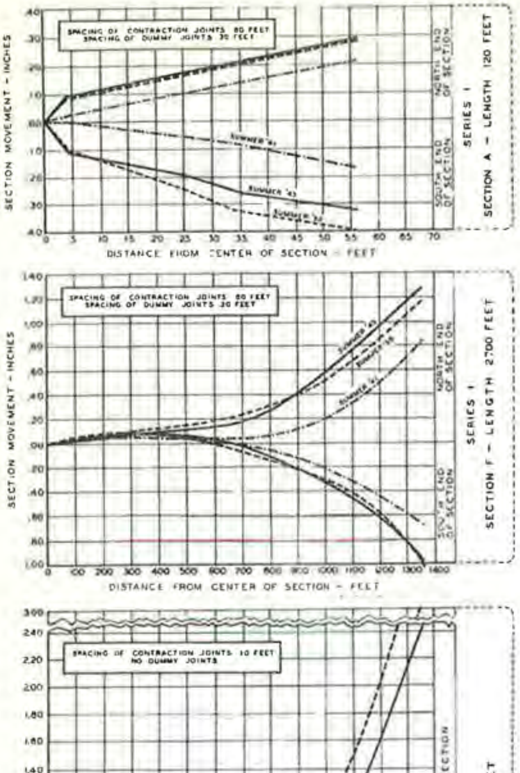


Figure 16. Relation between section movement and distance from center of section.

about 1800 feet, it may be expected that every point in each half of the section will display some movement with respect to the center of the section. For short sections such as illustrated by the graph at the top of Figure 16, (Series 1A) the movement of any point in either half of the section is approximately proportional to its distance from the center of the section.

It may be noted further that movements of the ends of each section are quite similar in character. In some instances certain inherent construction features, such as horizontal or vertical alignment, soil conditions, and bridge structures, no doubt influence the relative movement of the entire section, resulting in a general displacement of the whole section toward the right or left causing the point of zero movement to occur on either side of the geometric center of the section. It is also indicated in Figure 16 that the sections experienced their greatest movement during the first five years after construction.

Summary

The study of expansion and contraction joint movement has brought out several interesting and significant facts concerning slab behavior under varying expansion and contraction joint spacing. (1) The seasonal movements of the expansion joints indicate that there takes place during the first year after construction a considerable expansion and subsequent permanent displacement of the slab ends, using at least 50 percent of the space originally provided. (2) Subsequent to the first year's movement the section ends oscillate with seasonal climatic changes and the amplitude of these seasonal movements gradually diminishes with time. A slow, progressive permanent displacement also takes place which is greatest during the first 5 to 6 years and levels off thereafter. Eventually the joint filler will become compressed to such a state that no further longitudinal movement can occur. (3) Contraction joint spacing has considerable influence upon the amplitude of expansion joint movement. (4) All contraction joints acquire a small permanent opening which increases with time. The degree of joint movement and amount of residual opening is more pronounced as the distance between contraction joints is increased. (5) The movement of contraction joints is greater near the expansion joints than it is near the center of the sections. (6) Dummy joints react similarly to contraction joints but to a much smaller degree. (7) In sections of pavement greater than 1800 feet in length without expansion joints, there is a point of zero longitudinal extension approximately 700 to 900 feet from the ends of the section. Consequently, the central portion of such sections at elevated temperatures will be under restraint similar to that of continuous slabs in which no expansion joints have been provided.

TABLE 7

SUMMARY OF PAVEMENT CRACKING AND JOINT SPALLING IN RELATION TO JOINT SPACING

Section No.	Exp. Joint Spacing ft.	Series 1 9"-7"-9" 30 Foot Slabs 93 lb. Steel: Total Slabs	No. Cracked	Percent Cracked	Cracking, ft.: Transverse	Diagonal	Longitudinal	Total	Series 2 9"-7"-9" 15 Foot Slabs 27 lb. Steel: Total Slabs	No. Cracked	Percent Cracked	Cracking, ft.: Transverse	Diagonal	Longitudinal	Total
A	120	12	1	8	7	0	0	7	24	0	0	0	0	0	0
B	240	24	0	0	0	0	0	0	47	0	0	0	0	0	0
C	480	48	0	0	0	0	0	0	96	0	0	0	0	0	0
D	900	61	3	5	40	0	0	40	120	3	2	66	0	0	66
E	1800	63	4	7	39	0	6	45	120	0	0	0	0	0	0
F	2700	89	13	14	167	0	0	167	179	0	0	0	0	0	0
Summary		295	23	8	253	0	6	259	586	3	1	66	0	0	66

Section No.	Exp. Joint Spacing ft.	Series 3 9"-7"-9" 20 Foot Slabs No Steel: Total Slabs	No. Cracked	% Cracked	Cracking, ft.: Transverse	Diagonal	Longitudinal	Total	Series 4 9"-7"-9" 10 Foot Slabs No Steel: Total Slabs	No. Cracked	Percent Cracked	Cracking, ft.: Transverse	Diagonal	Longitudinal	Total
A	120	18	0	0	0	0	0	0	36	0	0	0	0	0	0
B	240	36	0	0	0	0	0	0	72	0	0	0	0	0	0
C	480	72	4	6	53	0	0	53	144	0	0	0	0	0	0
D	900	90	4	4	34	0	0	34	180	0	0	0	0	0	0
E	1800	90	4	4	11	10	35	56	180	0	0	0	0	0	0
F	2700	135	3	2	30	0	0	30	270	0	0	0	0	0	0
Summary		451	15	3	128	10	35	173	882	0	0	0	0	0	0

Spalling of Concrete at Joints

Section No.	Exp. Joint Spacing ft.	Series 1: No. Joints	No. Spalled	Percent Spalled	No. Spalled Areas	Average No. Spalls per Joint	Series 2: No. Joints	No. Spalled	Percent Spalled	No. Spalled Areas	Average No. Spalls per Joint
A	120	12	0	0	0	0.0	24	1	4	1	1.0
B	240	24	6	25	7	1.1	47	1	2	1	1.0
C	480	48	5	10	11	1.7	96	12	13	18	1.4
D	900	60	13	19	26	2.0	120	27	25	32	1.2
E	1800	63	22	35	70	1.3	120	18	15	19	1.1
F	2700	90	34	38	40	1.6	179	19	11	26	1.4
Summary		294	82	28	122	1.5	586	79	13	97	1.2

Section No.	Exp. Joint Spacing ft.	Series 3: No. Joints	No. Spalled	Percent Spalled	No. Spalled Areas	Average No. Spalls per Joint	Series 4: No. Joints	No. Spalled	Percent Spalled	No. Spalled Areas	Average No. Spalls per Joint
A	120	28	1	14	4	1.0	36	1	3	1	1.0
B	240	36	2	8	2	1.0	72	5	7	5	1.0
C	480	72	20	26	72	1.1	144	12	8	15	1.2
D	900	90	16	18	17	1.1	180	5	4	10	1.2
E	1800	90	32	36	35	1.1	180	3	2	3	1.0
F	2700	135	13	10	13	1.2	270	12	4	11	1.1
Summary		451	87	19	95	1.1	882	41	5	55	1.3

STRUCTURAL PERFORMANCE IN RELATION TO JOINT SPACING

The total length of cracking which has occurred in Series 1 to 4, whose slab lengths are 30, 15, 20, and 10 feet respectively, has been summarized in Table 7 for comparative study. These data, when summarized in the following manner, show an interesting relationship between slab length and total cracking.

Series	Slab Length	Cracking in feet: Transverse	Diagonal	Longitudinal	Total
1	30	253	0	6	259
3	20	128	10	35	173
2	15	66	0	0	66
4	10	0	0	0	0

The longitudinal and diagonal cracking listed under Series 3 has occurred in Section E. In this section, mechanical load transfer devices were omitted at transverse joints. The 10 feet of diagonal cracking listed under Series 3E represents the only corner break which has developed on the Design Project to date. The corner break is in the northbound lane at Station 100+70. The 35 feet of longitudinal cracking in Series 3E has occurred between Stations 1016+10 and 1016+50. The cracks have developed approximately 3 feet in from pavement edge on both sides of pavement.

Condition of Concrete

In general, the concrete surface in Series 1 to 4 is in excellent condition. Very little surface scaling has developed except for Section E in Series 4. Considerable scaling has appeared at joints. Records indicate that this is due to poor construction practice and finishing. However, a considerable amount of spalling has started to develop along joint edges. The extent to which spalling has developed to date will be found in Table 7. For comparative study, the spalling data from Table 7 has been summarized below.

Series	Slab Length ft.	% of Joints Spalled	Number of Spalled Areas
1	30	28	122
3	20	19	95
2	15	13	97
4	10	5	55

PAVEMENT ROUGHNESS IN RELATION TO JOINT SPACING

In September, 1941, and again in August, 1949, a series of pavement surface roughness tests were conducted on the entire test road by personnel of the Bureau of Public Roads using their specially designed machine constructed to record the number of surface irregularities in a definite distance. The study was made primarily to compare the riding qualities of various sections of the pavement, especially of those having varied expansion and contraction joint spacing, and to determine change in roughness with time.

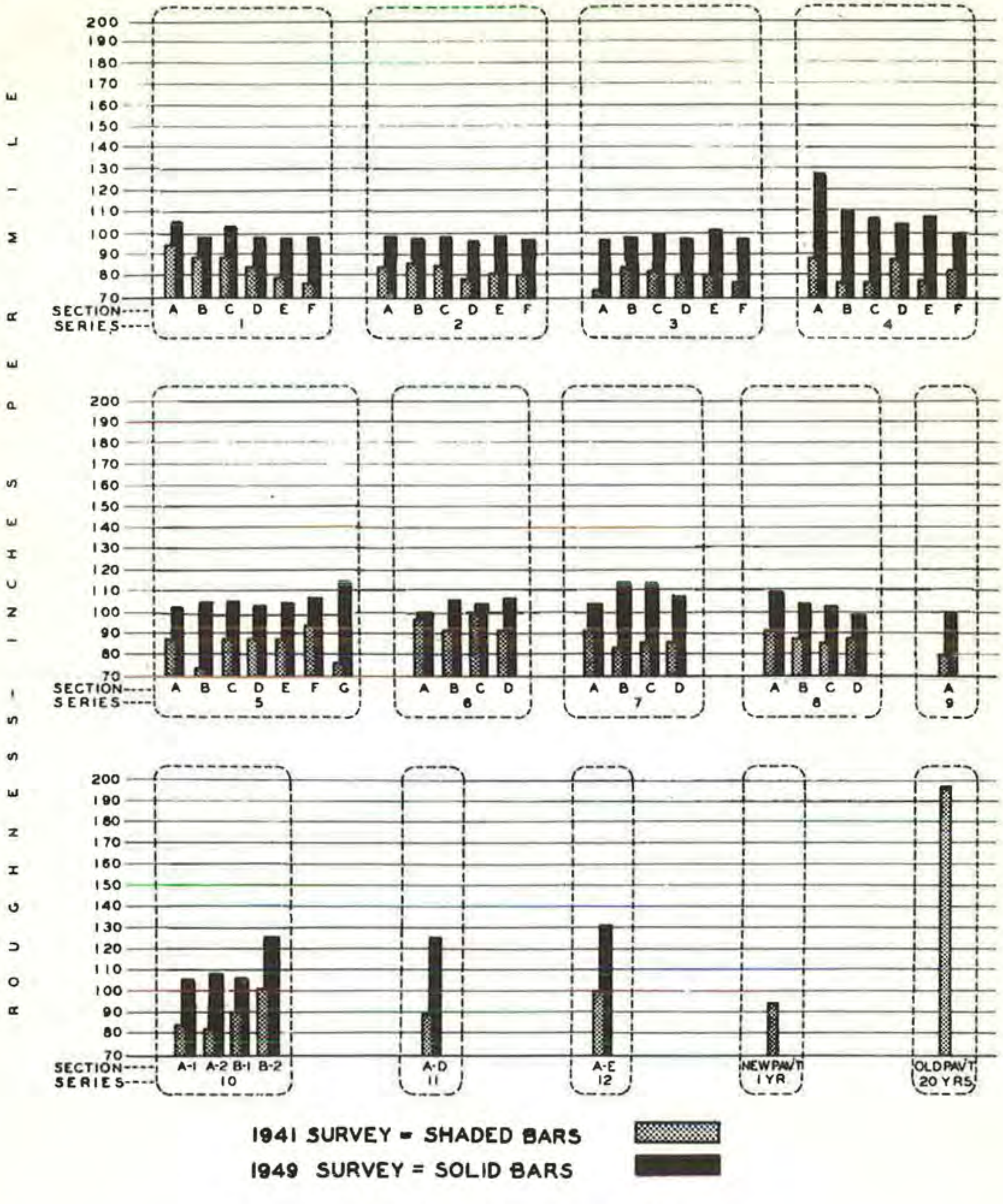


Figure 17. Road roughness data.

In conducting the tests each section of each series was taken as an increment to compare the surface roughness condition for the different joint spacings and concrete mixture variations. In 1941, additional tests were made on nearby projects, one of the same age as the test road and another project 20 years old, to afford a comparison with standard construction and to determine an expectant roughness factor. The roughness data obtained from the two series of tests are presented graphically in Figure 17. The original roughness factor for the entire project ranged from 73 to 101 units per mile, an indication, in general, of good workmanship and excellent riding qualities.

Considering first the results of the 1941 roughness tests, it is indicated that in the early life of the test road the roughness factor has no significant relation to joint spac

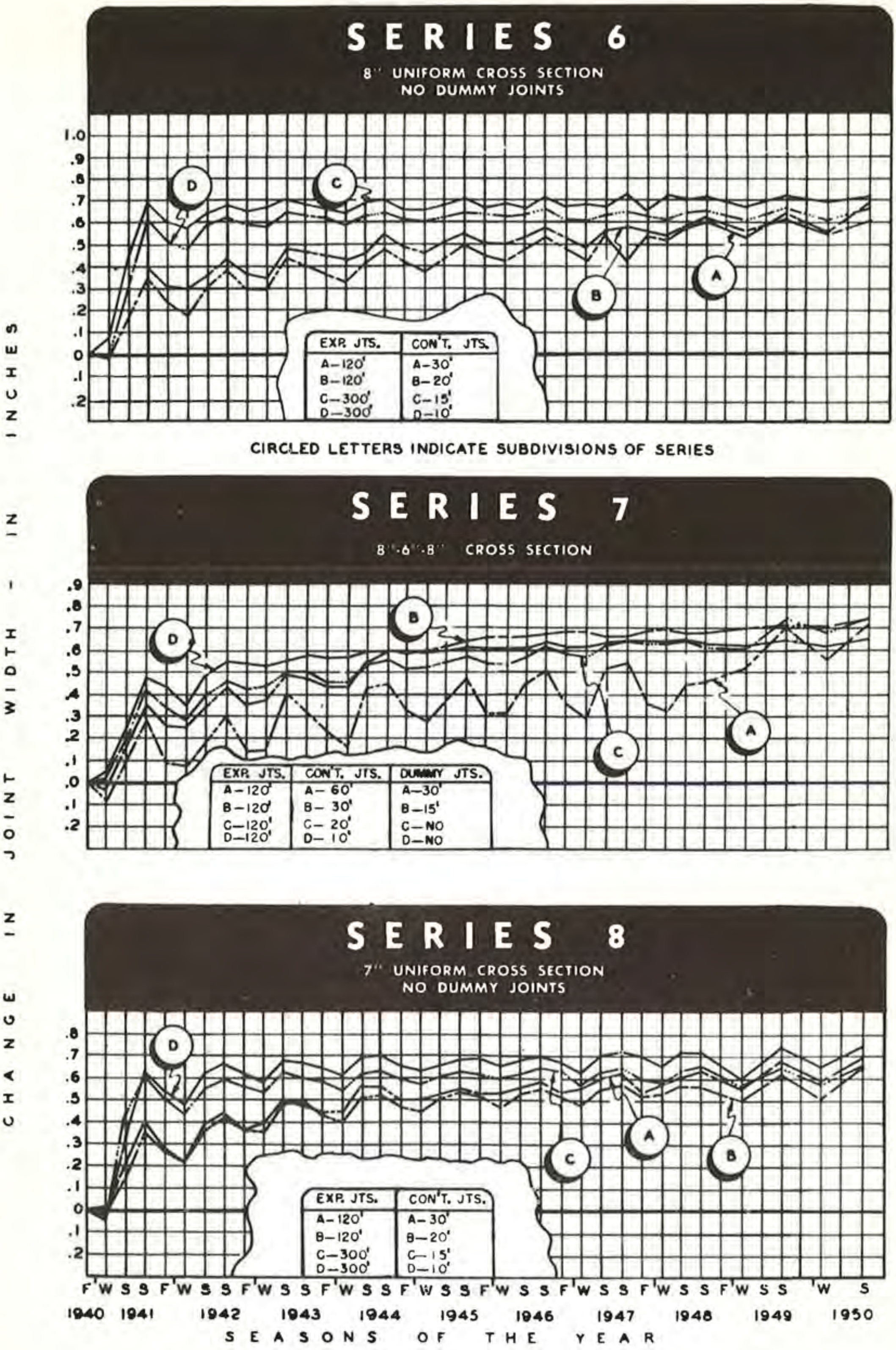
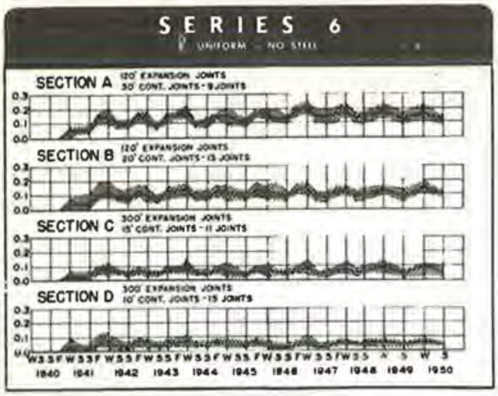
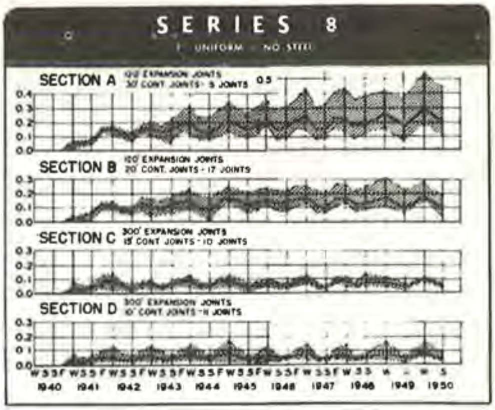


Figure 18. Seasonal changes in expansion joint width.





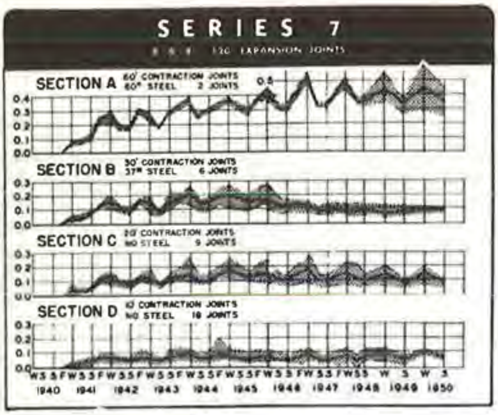


Figure 19. Seasonal changes in contraction joint width. Change in joint width, in inches.

ing, especially when good workmanship is attained. It is evident that Series 3 and 4, with 20- and 10-foot contraction joint spacing, have slightly lower average roughness factors than those of Series 1 and 2 with greater contraction joint spacing.

The 1949 roughness tests show that all sections have increased in roughness approximately to the same degree. However, Series 4 with the 10-foot joint spacing had the greatest increase in roughness. The general increase in roughness for the entire Design Project is approximately 19 percent.

PAVEMENT PERFORMANCE IN RELATION TO CROSS SECTION

Four different types of pavement cross sections were included in the Design Project for the purpose of studying such factors as load capacity of subgrade versus slab thickness and the balanced or thickened edge cross section versus equivalent uniform thickness. The cross sections set up for study include the 9-7-9-inch, and its approximate equivalent 8-inch uniform; the 8-6-8-inch, and its approximate equivalent 7-inch uniform. The portions of the Design Project devoted to this study included certain sections of Series 1-2-3 and 4 and Series 6, 7, and 8.

In general, nothing of note has developed so far in any of the series involved from which conclusive data can be established. The joints and slabs in all sections have, after ten years, begun to show marked difference in their relative behavior due to normal service conditions. The study emphasizes how very important it is to exercise rigid examination, inspection, and control over the preparation of subbase and subgrades for concrete pavement construction.

Expansion and Contraction Joint Movement

In Series 6, 7, and 8 expansion and contraction joint spacing were considered in conjunction with cross section design. The expansion joints are spaced at 120 feet and

300 feet with contraction joints at 30, 20, 15, and 10 feet. Also the sections 6 and 8 and part of section 7 were not reinforced with steel mesh.

The joint width measurements at the present time, as graphically presented in Figures 18 and 19 indicate no significant relationship between joint movement and thickness or shape of pavement cross section. The joint movements in Series 6, 7, and 8 duplicate very closely the magnitude, annual amplitude and progressive displacement of the joints located in various series containing similar expansion and contraction joint spacing.

Physical Condition of Sections

In the winter following the construction of the test road some cracking developed in part of Series 6 from Station 90+70 to 94+30 and 97+60 to 103+00. During the first four years a considerable number of longitudinal and transverse cracks continued to develop in this area as illustrated in Figure 20.

The crack pattern throughout the cracked area is typical of that caused by heaving of a rigid pavement due to volume changes in the subgrade. The pavement in Series 6 was constructed on a 12-inch sand subbase overlying Emmet loamy sand and Isabella loam. A soils survey in the spring of 1944 revealed several factors contributing to the abnormal cracking. In the first place the sandy clay subgrade material was badly rutted, protruding into the subbase material practically the full depth in some places. Such a condition would naturally prevent the normal lateral drainage of the subbase material; consequently, water pockets formed directly beneath the slab, which resulted in the longitudinal cracking of the slab.

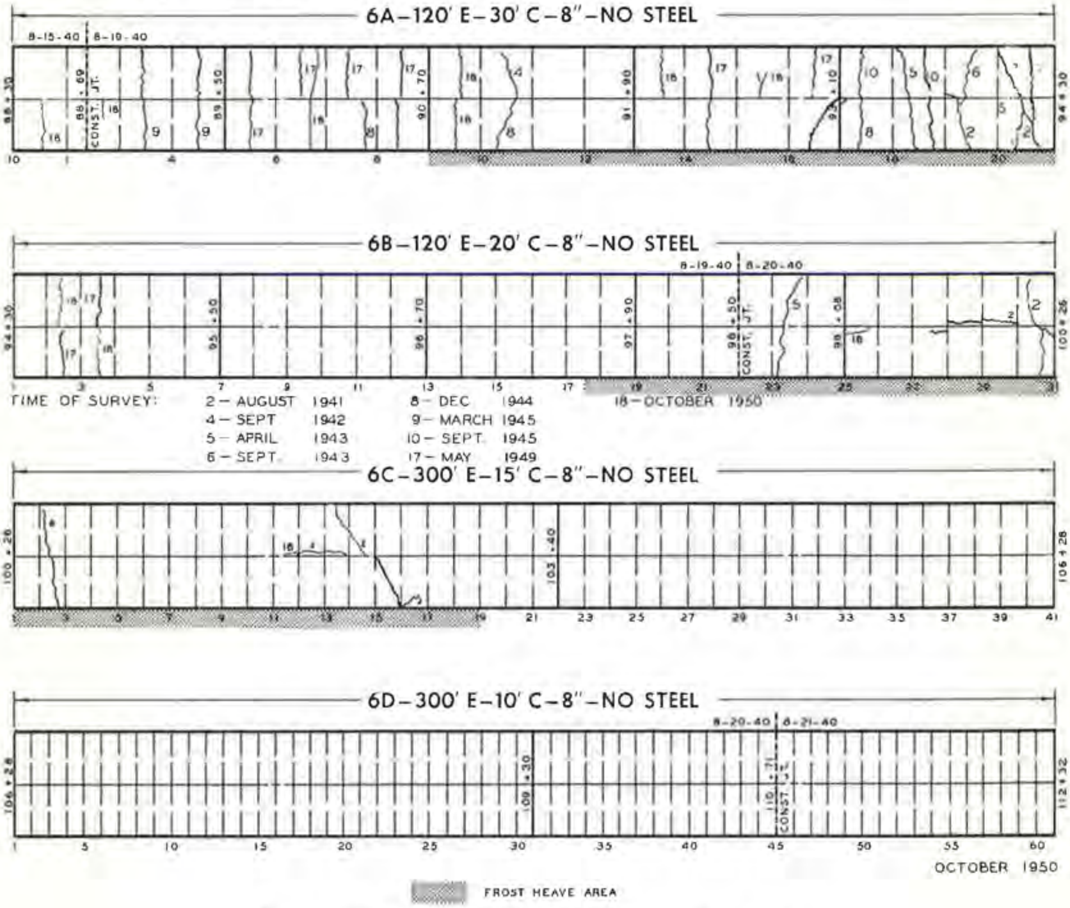

6A–120' E–30' C–8"–NO STEEL
8-15-40
8-19-40
88+30
88+69 CONST. JT.
89+50
90+70
91+90
93+10
94+30
6B–120' E–20' C–8"–NO STEEL
8-19-40
8-20-40
94+30
95+50
96+70
97+90
98+50 CONST. JT.
98+68
100+26
TIME OF SURVEY:
2 – AUGUST 1941
4 – SEPT 1942
5 – APRIL 1943
6 – SEPT. 1943
8 – DEC 1944
9 – MARCH 1945
10 – SEPT. 1945
17 – MAY 1949
18 – OCTOBER 1950
6C–300' E–15' C–8"–NO STEEL
100+26
103+40
106+28
6D–300' E–10' C–8"–NO STEEL
8-20-40
8-21-40
106+28
109+30
110+71 CONST. JT.
112+32
OCTOBER 1950
FROST HEAVE AREA

Figure 20. Condition of pavement in Series 6.

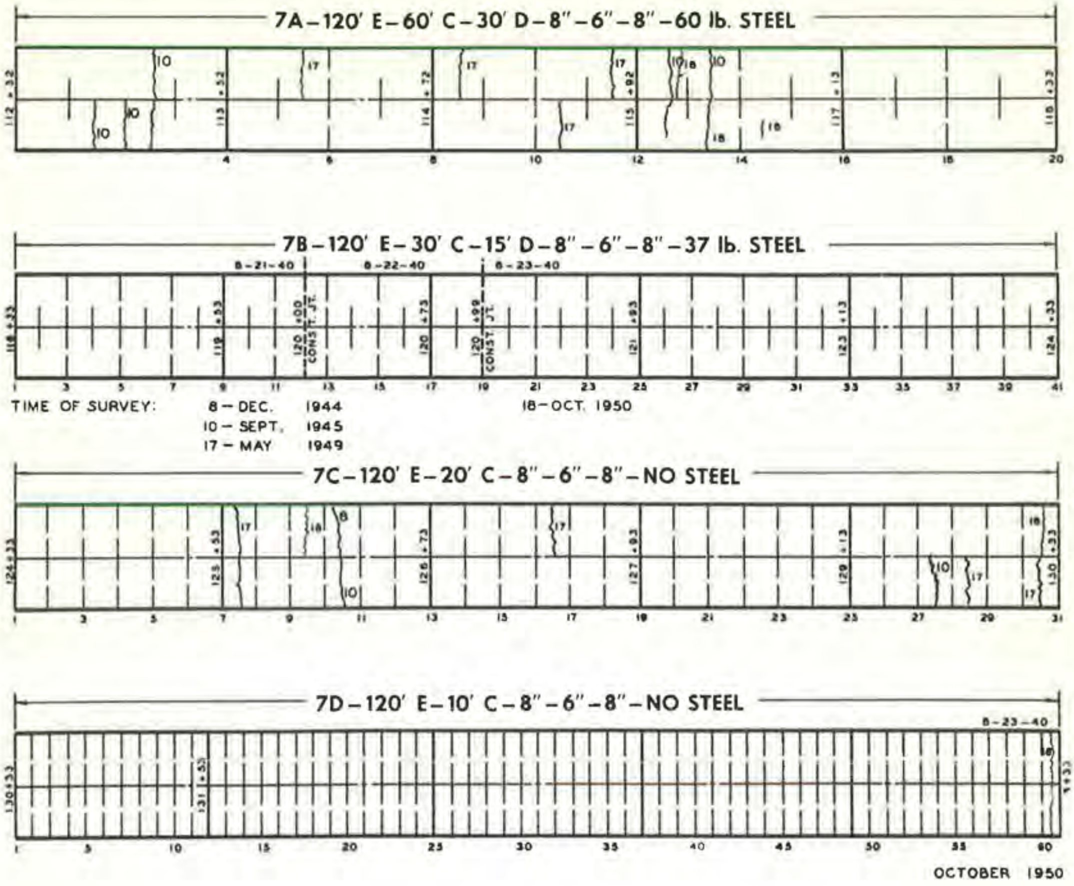

7A–120' E–60' C–30' D–8"–6"–8"–60 lb. STEEL
7B–120' E–30' C–15' D–8"–6"–8"–37 lb. STEEL
TIME OF SURVEY:
8–DEC. 1944
10–SEPT. 1945
17–MAY 1949
18–OCT. 1950
7C–120' E–20' C–8"–6"–8"–NO STEEL
7D–120' E–10' C–8"–6"–8"–NO STEEL
OCTOBER 1950

Figure 21. Condition of pavement in Series 7.

In addition to the faulty subbase condition it was learned that the subgrade material in the cracked sections contained pockets and laminations of peat, clay and silt, all of which undergo considerable volume change during freezing and thawing of the subgrade. Ice lenses were readily discernible in soil samples from test holes. Such subgrade and subbase soil conditions are no doubt responsible for the preponderance of transverse and diagonal cracks within these areas.

Transverse cracks are beginning to appear, however, in other areas of Series 6. In Figure 20 it will be noted that all but two of the 20 slabs in Section 6A have developed transverse cracks.

In Series 7, with 8-6-8-inch cross section, the first transverse crack was noted in December, 1944 and, since that time, several more have developed as may be seen in Figure 21.

Only two transverse cracks have developed in Series 8 with 7-inch slab thickness. The first was observed in April, 1943 and the last in December, 1944 (see Figure 22).

A review of Figure 1 will show that Series 8 lies entirely on an excellent granular subgrade soil whereas all of Series 6 and 7, with the exception of Section 7D, were placed on a subbase over a questionable subgrade material. This is no doubt the reason why Series 8 has preformed so well over the past years. A complete crack and spall summary will be found in Table 8. For comparative purposes the crack information in Table 8, for Series 6, has been tabulated both for the entire series and also just for those slabs outside of frost heave areas previously described.

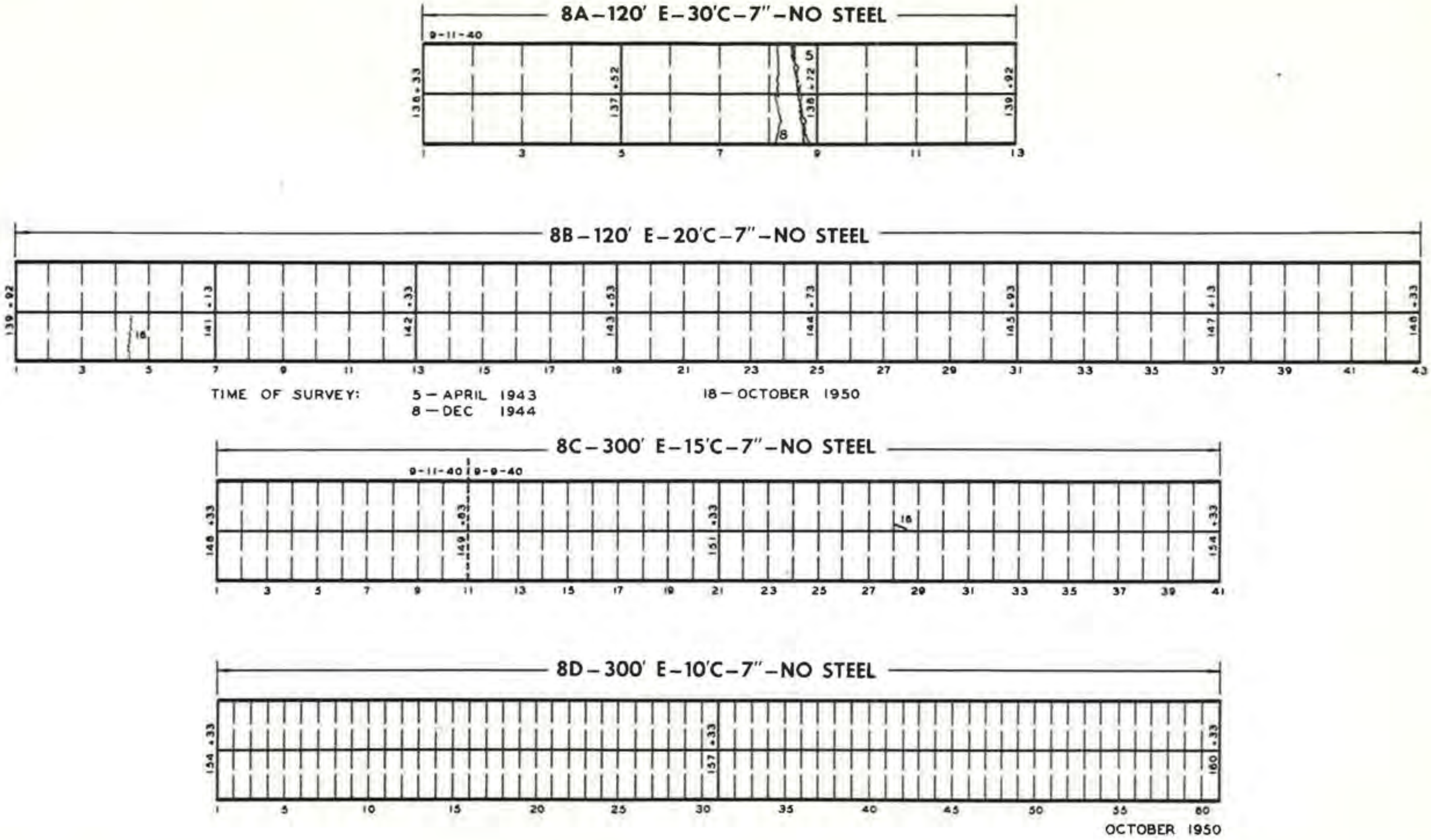

8A–120' E–30'C–7"–NO STEEL
9-11-40
136+33
137+52
138+72
139+92
5
8
1
3
5
7
9
11
13
8B–120' E–20'C–7"–NO STEEL
139+92
141+13
142+33
143+53
144+73
145+93
147+13
148+33
18
1
3
5
7
9
11
13
15
17
19
21
23
25
27
29
31
33
35
37
39
41
43
TIME OF SURVEY:
5 — APRIL 1943
8 — DEC 1944
18 — OCTOBER 1950
8C–300' E–15'C–7"–NO STEEL
9-11-40
9-9-40
148+33
149+83
151+33
154+33
18
1
3
5
7
9
11
13
15
17
19
21
23
25
27
29
31
33
35
37
39
41
8D–300' E–10'C–7"–NO STEEL
154+33
157+33
160+33
1
5
10
15
20
25
30
35
40
45
50
55
60
OCTOBER 1950

Figure 22. Condition of pavement in Series 8.

TABLE 8

SUMMARY OF PAVEMENT CRACKING AND JOINT SPALLING IN RELATION TO CROSS SECTION

			Series 6 – 8" Uniform No Reinforcement														Series 7 – 8"-6"-8" Cross Section Reinforcement – 60 lb. – 37 lb. – None									Series 8 – 7" Uniform No Reinforcement						
									Cracking, ft.												Cracking, ft.								Cracking, ft.			
Section	Slab Length	Exp. Joint Spacing	No. Slabs		No. Slabs Cracked		Percent Cracked		Transverse		Diagonal		Longitudinal		Total Cracking	Normal Cracking[a]	Exp. Joint Spacing	No. Slabs	No. Cracked	Percent Cracked	Transverse	Diagonal	Longitudinal	Total	Reinforcing lb./100 sq. ft.	No. Slabs	No. Slabs Cracked	Percent Cracked	Transverse	Diagonal	Longitudinal	Total Cracking
			Total[a]		Total[a]		Total[a]		Total[a]		Total[a]		Total[a]																			
A	30	120	20	8	18	8	90	100	322	139	111	0	4	0	437	139	120	20	9	45	146	0	0	146	60	12	1	8.3	22	23	0	45
B	20	120	30	16	8	2	27	12	33	48	38	0	62	0	133	48	120	40	0	0	0	0	0	30	37	42	1	2.4	11	0	0	11
C	15	300	40	22	7	0	17	0	0	0	93	0	48	0	141	0	120	30	7	23	110	0	0	110	None	40	1	2.5	0	3	0	3
D	10	300	60	0	0	0	0	0	0	0	0	0	0	0	0	0	120	60	1	2	22	0	0	22	None	60	0	0	0	0	0	0
Summary			150	46	33	10	21	30	355	137	242	0	114	0	711	187		150	17	11	278	0	0	308		154	3	2	33	26	0	59

								Spalling									
Section No.	Slab Length	Exp. Joint Spacing	Number Joints	Number Spalled	Percent Spalled	Number Spalled Areas	Average per Joint	Number Joints	Number Spalled	Percent Spalled	Number Spalled Areas	Average per Joint	Number Joints	Number Spalled	Percent Spalled	Number Spalled Areas	Average per Joint
A	30	120	20	6	30	8	0.4	20	2	10	3	0.2	12	5	41	9	0.8
B	20	120	30	13	43	17	0.6	40	2	5	2	0.1	42	11	24	12	0.3
C	15	300	40	25	61	31	0.8	30	2	7	2	0.1	40	11	25	13	0.3
D	10	300	60	15	25	16	0.3	60	0	0	0	0.0	60	7	12	8	0.1
Summary			150	59	39	75	0.5	150	6	4	7	0.1	154	34	22	42	0.4

[a] Slab cracking in sections outside of frost heave areas. – 6A – Stations 90+70 to 94+30, 6B – Stations 97+60 to 100+26, 6C – Stations 100+26 to 103+00.

CRACK AND SPALL DATA IN RELATION TO SLAB LENGTH

		Slab Length in feet																	
		30'				20'				15'				10'					
		A				B				C				D					
Series	Slab Thickness	Total Cracking ft.	Total Slabs	Percent Slab Cracks	Spalled Joints %	Total Cracking	Total Slabs ft.	Percent Slabs Cracked	Spalled Joints %	Total Cracking	Total Slabs ft.	Percent Slabs Cracked	Spalled Joints %	Total Cracking ft.	Total Slabs	Percent Slabs Cracked	Spalled Joints %	Total Cracking ft.	Percent Spalled Joints
8	7"	45	12	8	41	11	42	2	24	3	40	3	25	0	60	0	12	59	22
7	8"-6"-8"	145	20	45	10	0	40	0	5	110	30	23	7	22	60	2	0	308	4
6[a]	8"	139	8	100	30	48	16	12	43	0	22	0	61	0	60	0	25	187	39
1-A	9"-7"-9"	7	12	8	0	-	-	-	-	-	-	-	-	-	-	-	-	7	6
3-A	9"-7"-9"	-	-		-	0	18	0	14		-	-		-	-	-	-	0	
2-B	9"-7"-9"	-		-	-	-	-		-	0	47	0	2	-	-	-	-	0	
4-B	9"-7"-9"	-	-	-	-	-	-	-			-	-	-	0	72	0	7	0	

[a] Slabs in frost heave areas Station 90+70 to 94+30 in Series 6A. Stations 97+60 to 100+26 in Series 6B. Stations 100+26 to 103+00 in Series 6C, not included in data.

Pavement Roughness

With reference to Figure 17, it may be seen that the riding qualities of Series 6, 7, and 8 are approximately the same with Series 7 being probably slightly rougher. All three series have increased approximately the same amount with age.

Summary

All other factors considered equal, the data presented above show a definite relationship between joint spacing and pavement performance at least up to slab lengths of 30 feet. It is indicated that within the limitations of the study, transverse cracking and joint spalling increases with the increase in joint spacing but roughness will increase with decrease in slab length.

PAVEMENT PERFORMANCE IN RELATION TO STEEL REINFORCEMENT

Consideration was given to the problem of designing pavements with and without steel reinforcement. To this end different sections of the Design Project were constructed with 60, 37, and 0 pounds of steel reinforcement per 100 sq. ft. of pavement. The problem of reinforcement was also considered in connection with contraction and dummy joint construction as well as in the construction of continuous slabs of varying lengths without intermediate contraction or dummy joints.

Reinforcement in Relation to Dummy Joint Construction

In Series 2, containing the 37 lb. reinforcement it is evident, through the sudden occurrence of abnormally large joint width readings, that the steel has failed at certain dummy joints. Of 4 dummy joints at the west end of Series 2E, two have opened excessively for the first time in the winter of 1949, these are joint 108 at Station 907+70 and joint 112 at Station 908+30. Of seven dummy joints in Series 2B, 3 have opened excessively at various times as follows:

Joint 33 at 949+25	Winter 1948
Joint 39 at 950+15	Winter 1949
Joint 41 at 950+45	Winter 1950

Similar conditions have not occurred in dummy joints in Series 1 containing 60 lb. steel. No other comparative data are available for judging merits of different amounts of reinforcement.

Continuous Slabs with and without Reinforcement

Two sections, designated Series 11 and 12 of the Design Project, were constructed in conjunction with the Durability Project of the Test Road (see Table 1). These two sections of pavement included continuous slabs of different lengths with and without reinforcement. Series 11 and 12 were established in order to obtain more comprehensive data relative to concrete pavement design, especially in relation to the behavior of continuous slabs versus slabs with intermediate contraction and plane of weakness joints and for slabs constructed with and without reinforcement. Special attention has also been given to changes in slab length, progressive cracking of the slabs, and the influence of steel on degree and character of cracking. Each series contains continuous lengths 90, 120, 360, and 600 feet. Steel reinforcement at 60 pounds per 100 sq. ft. was placed in Series 11. Series 12 was not reinforced. It is to be noted that these sections were not built by the contractor of the Deisgn Project and the materials, such as cement and aggregates involved in the construction of the concrete slabs, were obtained from entirely different sources. It is believed that these factors will have very little effect upon the final results derived from the study.

Pavement Movement in Relation to Slab Length

Reference monuments were installed at the ends, center, and quarter points of the slabs in Series 11 and 12 to observe their movements over a period of years. Unfor-

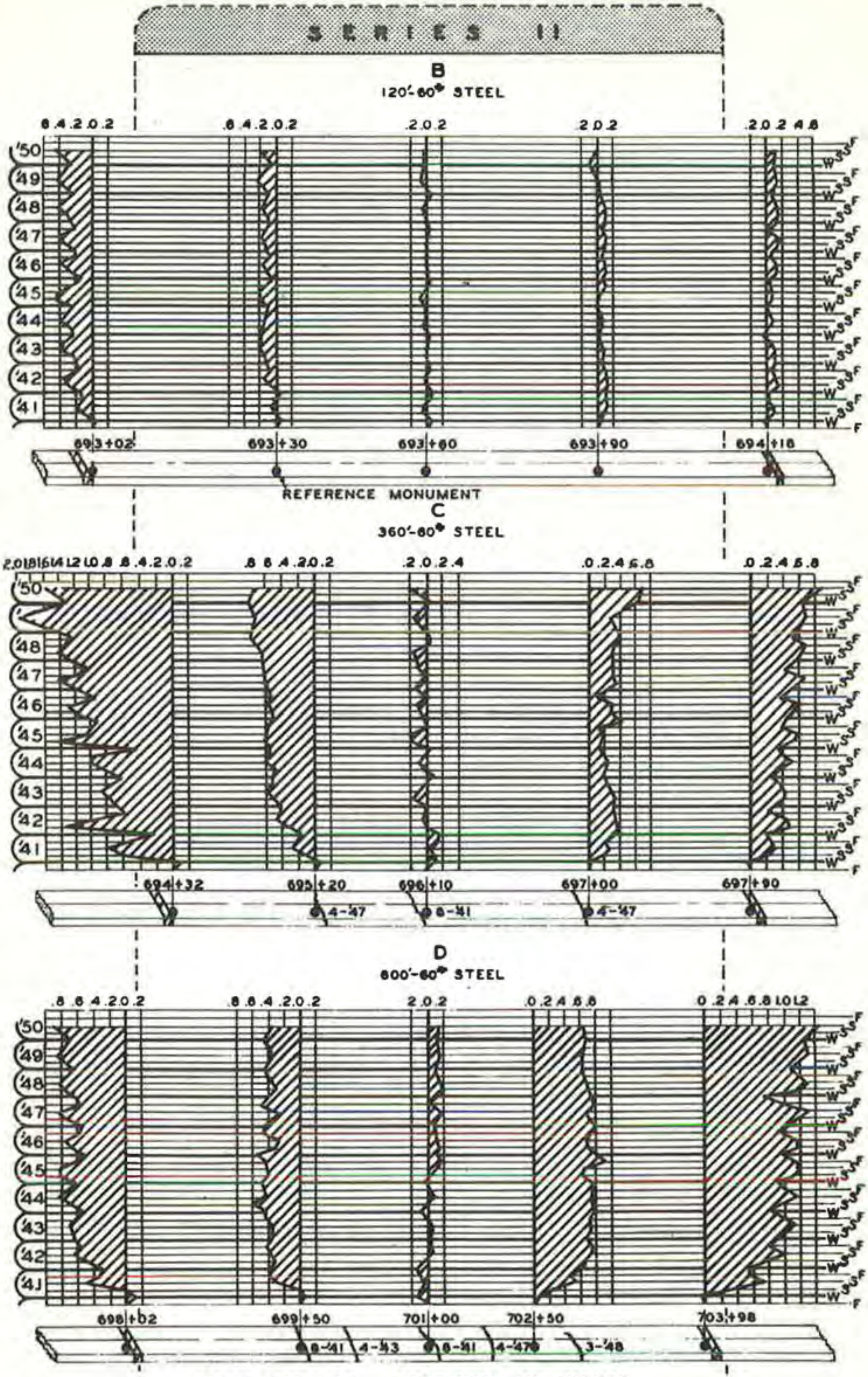


Figure 23. Seasonal changes in section length.

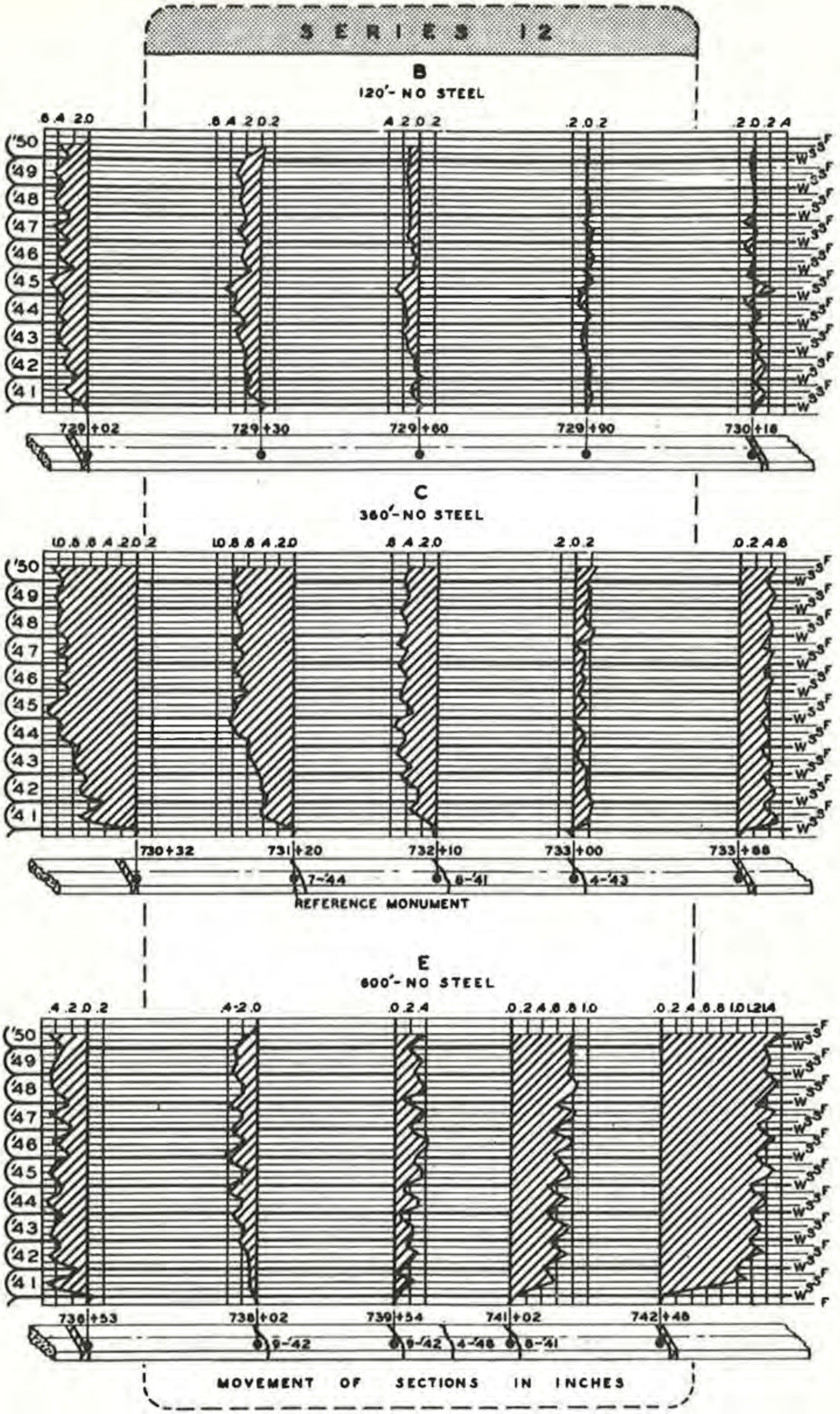


Figure 24. Seasonal changes in section length.

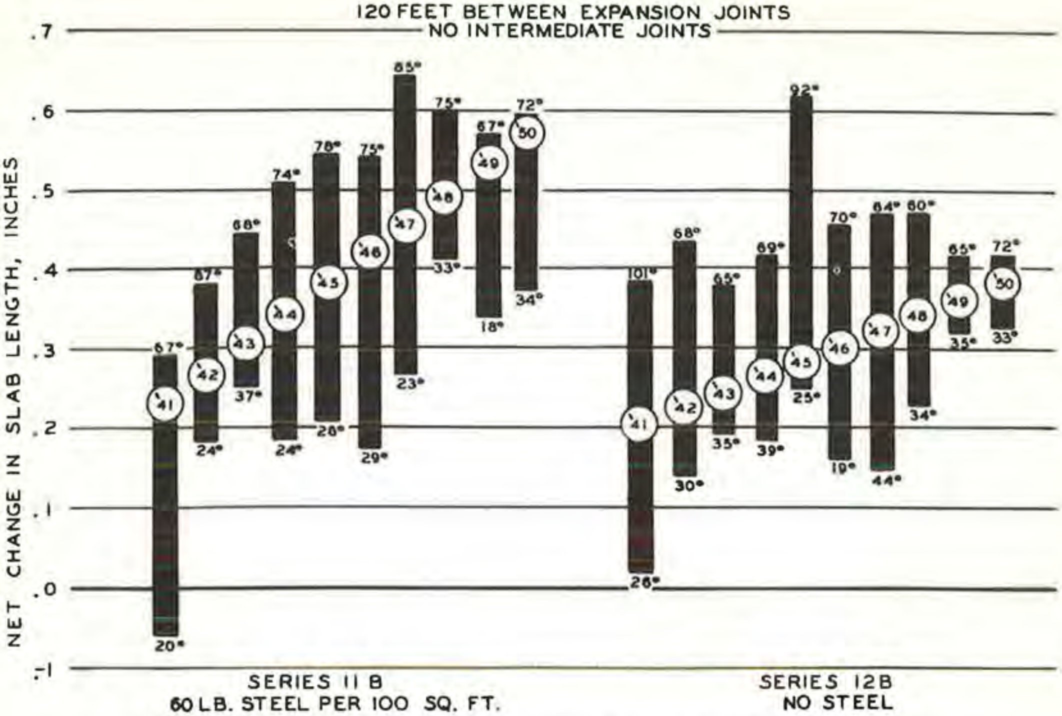


Figure 25. Annual and progressive changes in slab length.

tunately, the subsequent cracking of the long sections has reduced them to a series of short slabs which has restricted somewhat the original purpose of the study. Seasonal readings were continued, however, and the slab movement at the various monuments has been presented graphically in Figures 23 and 24. In general, the data further show an influence of restraint on slab movement similar to that observed on the 2700-foot sections in Series 1, 2, 3, and 4.

Since the 120-foot slabs of Sections 11B and 12B have not broken, it is possible to observe the net change in their length. The net seasonal and progressive changes for the 10-year period are shown in Figure 25. It may be observed from the plotted data in Figure 25 that both slabs have acquired a residual increase in length of approximately 0.3 to 0.4 of an inch. The greatest net change in slab length apparently took place during the first year, whereas during the succeeding years the increase in length has been gradual and much smaller in amount. The values plotted represent field measurements at the existing slab temperatures indicated for winter and summer seasons.

Physical Condition of Slabs

The physical condition of the slabs in both series with respect to cracking is shown in Figure 26. The crack pattern of both series is very similar. For example, the 120-foot sections have not cracked and the 360-foot and 600-foot sections of the two series are cracking in a similar manner. However, the full transverse cracks have formed sooner in the unreinforced section. Fortunately, both Series 11 and 12 have been constructed on the same type of subgrade soil identified as a sand of the Rubicon and Newton series.

The total length of cracking in Series 11C and 11D is 220 feet. In Series 12C and 12E, comparable sections to Series 11C and D, the total length of cracking is 237 feet.

With few exceptions, the cracks formed first at the monument boxes which were set in the slabs to measure slab movement. The maximum width of crack opening in the slabs varied from $\frac{1}{4}$ inch to $\frac{1}{2}$ inch or more. From visual examination and crack width measurements, it is apparent that the steel mesh in Series 11 has broken at the cracks

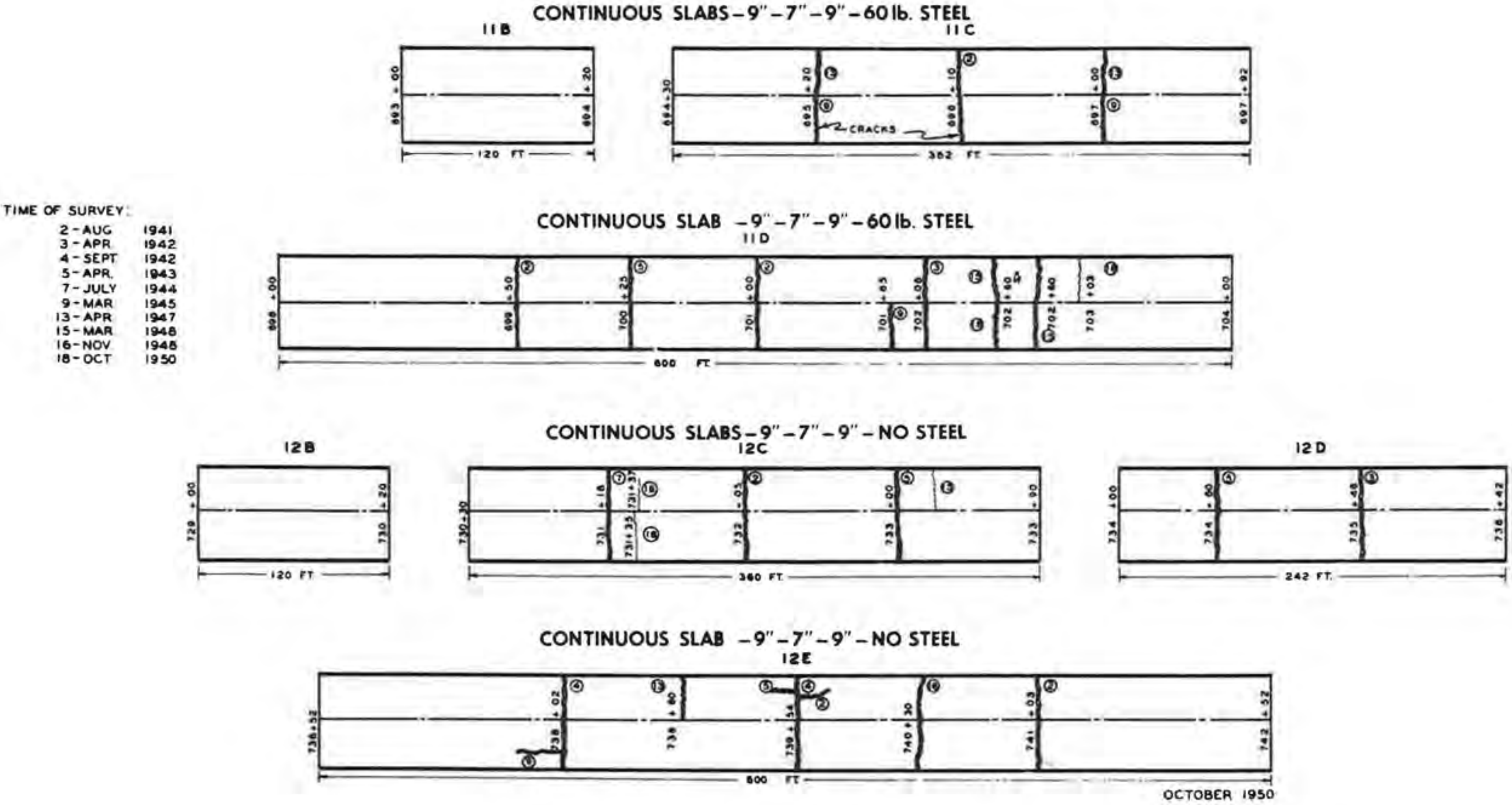

CONTINUOUS SLABS - 9" - 7" - 9" - 60 lb. STEEL
11 B
11 C
693 + 00
694 + 20
120 FT
694+30
695 + 20
696 + 10
697 + 00
697 + 92
CRACKS
382 FT
TIME OF SURVEY:
2 - AUG 1941
3 - APR 1942
4 - SEPT 1942
5 - APR 1943
7 - JULY 1944
9 - MAR 1945
13 - APR 1947
15 - MAR 1948
16 - NOV 1948
18 - OCT 1950
CONTINUOUS SLAB - 9" - 7" - 9" - 60 lb. STEEL
11 D
698 + 00
699 + 50
700 + 25
701 + 00
701 + 85
702 + 08
702 + 60
702 + 80
703 + 03
704 + 00
600 FT
CONTINUOUS SLABS - 9" - 7" - 9" - NO STEEL
12 B
12 C
12 D
729 + 00
730 + 20
120 FT
730+30
731 + 16
731+35
731+37
732 + 05
733 + 00
733 + 90
360 FT
734 + 00
734 + 80
735 + 46
736 + 42
242 FT
CONTINUOUS SLAB - 9" - 7" - 9" - NO STEEL
12 E
736+52
738 + 02
738 + 80
739 + 54
740 + 30
741 + 03
742 + 52
600 FT
OCTOBER 1950

Figure 26. Condition of pavement in Series 11 and 12.

It is of interest to note that the original test slabs of comparable lengths in both series have broken down into individual slabs of average length, of 91 and 72 feet respectively in Series 11C and 12C and for Sections 11D and 12E, slab length average 86 and 120 feet, respectively. However, individual slab lengths vary in both series from 17 to 152 feet. Summary of slab lengths will be found in Table 9.

TABLE 9

SUMMARY OF SLAB LENGTHS IN SERIES 11 AND 12

Number and Length of Individual Slabs in 1950				
	Original Slab Length 360 ft.		Original Slab Length 600 ft.	
	Steel	No Steel	Steel	No Steel
Section	11C	12C	11D	12D
	92	90	120	149
	90	95	20	73
	90	70	52	76
	90	17	108	152
	Avg. 91	88	75	150
		Avg. 72	75	Avg. 120
			150	
			Avg. 86	

Riding Qualities

With reference to Figure 17, it will be noted that Series 11 and 12 have riding qualities comparable to the test sections located throughout the Design Project. However, Series 11 with the steel mesh, has a lower roughness factor than Series 12 without steel, which might be significant. Both series have increased materially in roughness over the past ten years.

Expansion and Contraction Joint Design

The design of a transverse joint necessitates consideration of structural features which will enable the joint to perform the function for which it is intended. It must provide for movement due to expansion or contraction, load transfer where necessary to maintain vertical alignment of abutting slabs, possess flexibility to permit warping, and provide adequate seal against infiltration of water and inert material.

Several units of various types of expansion and contraction joint construction in current practice were installed in the Design Project of the Test Road for comparative study. With respect to expansion joint construction two types of construction features were given major consideration; (1) the efficacy of standard dowel bar construction with fiber filler strip versus air chamber construction, and (2) different design features to maintain vertical alignment of abutting slabs. In the case of contraction joints special consideration has been given to the study of four outstanding construction features: (1) the sealed groove versus premolded bituminous fiber strip; (2) load transfer feature to maintain vertical alignment of the slabs; (3) features to insure proper formation of contraction joint; and (4) the feasibility of omitting load transfer devices in the case of short slabs and long spacing of expansion joints.

EXPANSION JOINT DESIGN

The following types of expansion joint construction have been considered:

Type DB-1 (1)	Highway Department Standard 1-inch bituminous fiber board with $\frac{3}{4}$- by 15-inch dowel bars at 15-inch spacing for load transfer.
Type TE (1)	Thickened edge slabs with 1-inch bituminous fiber board and one $1\frac{1}{4}$- by 18-inch dowel placed at each of the four corners, 9 inches in from the slab edge.
Type CB (2)	Uniform thickness slabs with 1-inch bituminous fiber board and one $1\frac{1}{4}$- by 18-inch dowel placed at each of the four corners, 9 inches in from slab edge.
Type A (2)	Standard 1-inch bituminous fiber board, but with no load transfer device.
Type DB-1 (3)	Air-chamber construction with 1-inch opening, top, bottom, and sides sealed with an asphalt-latex joint seal compound and using $\frac{3}{4}$- by 15-inch dowel bars at 15-inch spacing for load transfer.

Type TB (4)	Air-chamber construction with 1-inch opening sealed at bottom with an asphalt-latex joint seal compound and at top and sides with a premolded rubber seal. Translode base units were used as the load transfer feature.

Data presented previously in the report show that with few exceptions, such as in the stress curing section, the seasonal amplitudes of joint movement are comparable and that there appears to be a progressive narrowing of the joint widths, which seems to be a natural phenomenon in concrete pavements, especially those constructed with intermediate contraction joints or containing continuous slabs which have cracked.

Premolded Fiber Filler versus Air-Chamber

Joint width measurements and visual observations indicate that the premolded fiber

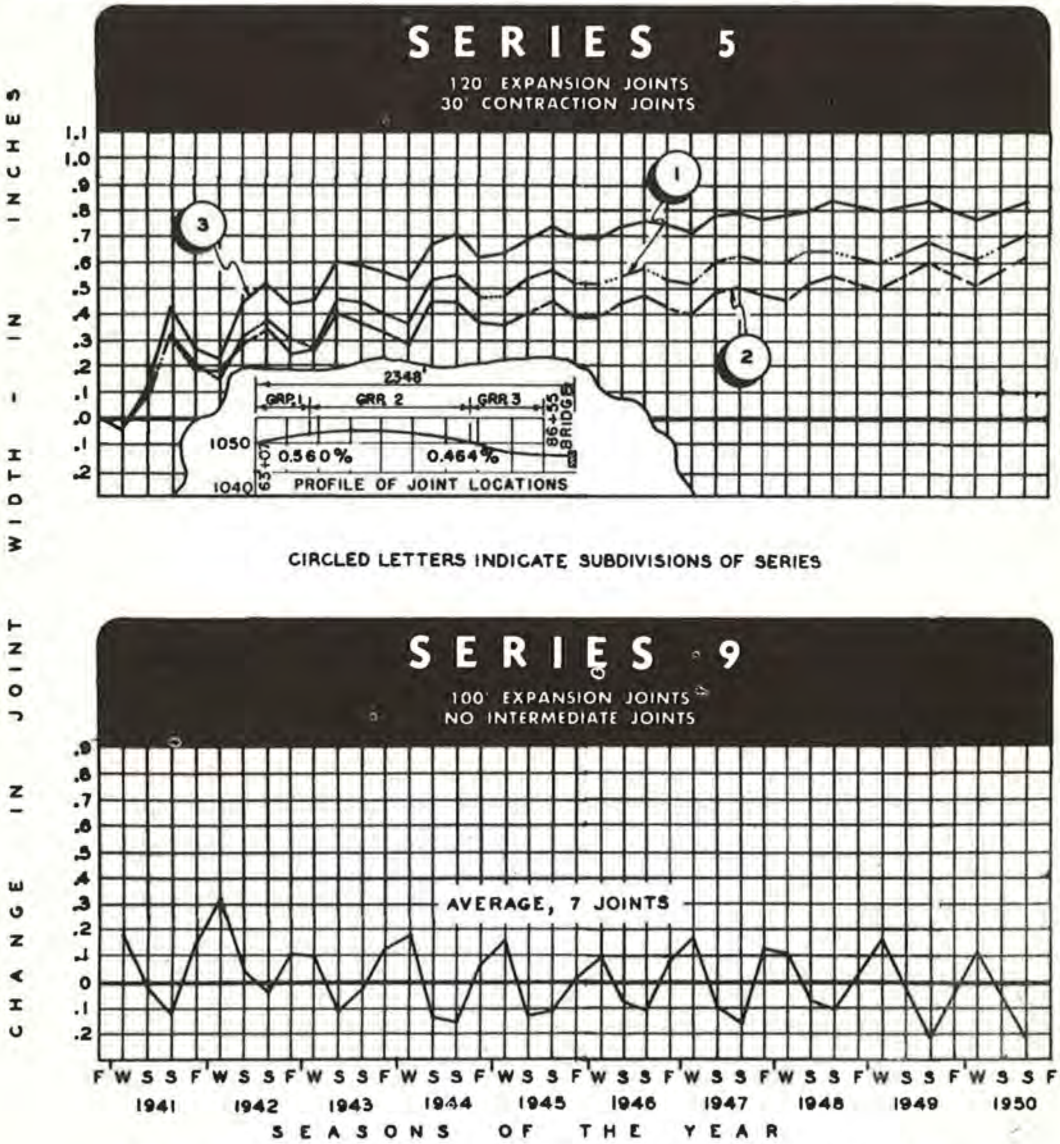


Figure 27. Seasonal changes in expansion joint width. Air chamber expansion joint.

board has certain advantages over the air-chamber type of joint. When the fiber board is properly installed, with respect to edge of joint and surface of the slab, it provides a good foundation for the bituminous sealer. Also, since it does not extrude it will remain intact to prevent infiltration of large aggregate pieces. Furthermore, it will distribute compressive forces due to expansion uniformly over the ends of the two abutting slabs. This latter function should decrease materially the inherent tendency of concrete pavements to disintegrate at expansion joints. Air-chamber expansion joints such as were included in the investigation require exceptionally good seals in order to perform their function satisfactorily. Since the two types of air-chamber expansion joints were constructed somewhat differently in different locations of the Design Project, their respective performances will be discussed separately.

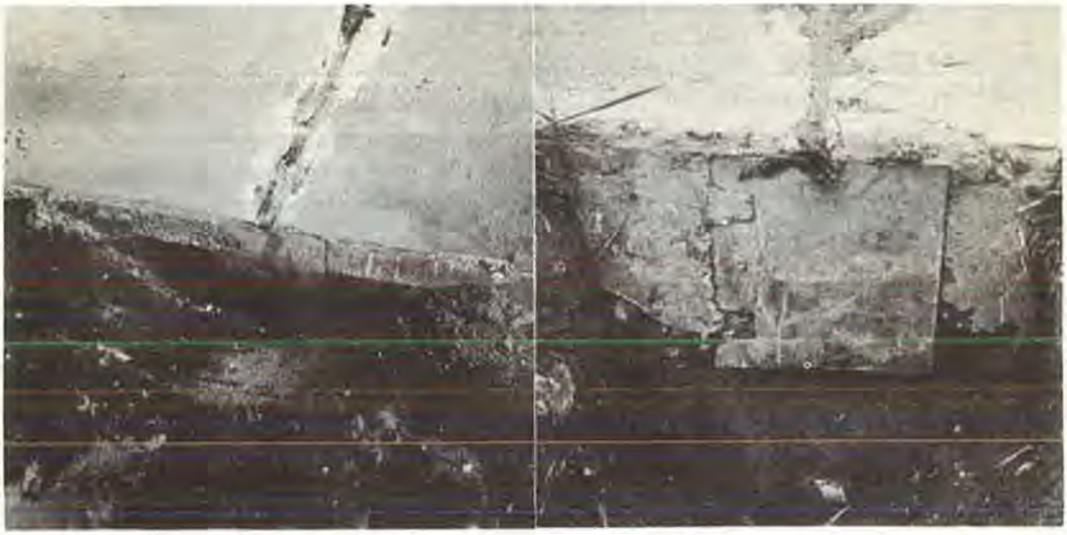


Figure 28. Air chamber expansion joint, Type DB-1 (3) Series 5, Station 79+60. (Left) at time of construction, 1940. (Right) 10 years later, 1950.

Type DB 1 (3) Series 5. Seasonal joint width changes for the two types of air-chamber expansion joints are presented graphically in Figure 27. In general, the joints in Series 5 have developed a slightly greater permanent closure than joints in other series constructed with premolded fiber. This is to be expected since these joints offer no restraint to the adjacent slabs. In the case of Type DB 1 (3) joints, this movement has caused excessive extrusion of the bituminous seal onto the pavement surface. In two cases in Section 5G the metal inserts employed to retain the bituminous seal in place have been pushed partially out of the joint. The abnormal movement encountered in Series 5, Sections F and G may be due to two constructional factors associated with this particular test section: (1) the two end sections F and G lie at the foot of a downgrade of 0.464 percent; (2) these two sections abut against a bridge structure which cannot shift laterally. In Figure 28 there are presented views of air-chamber expansion joint DB-1 (3) at the time of construction and 10 years later.

Type TB-(4) Series 9A. The air-chamber expansion joints used in Series 9A, Type TB-(4) have reacted very satisfactorily except for the premolded rubber seal which failed after two years in service. The premolded rubber seal through traffic action was pushed down into the joint about $\frac{3}{4}$ of an inch and rotated 90 degrees in the joints (see Figure 29). Eventually it was necessary to remove the rubber seal, thoroughly clean the joints and reseal. In the resealing process the old premolded rubber material was placed back into the joint opening at a depth of about $\frac{3}{4}$-inch and new hot-poured bituminous-rubber joint sealing compound was poured on top to effect the seal.

The average seasonal changes in joint width for Type TB-(4) expansion joints in Series 9A will be found in Figure 27.

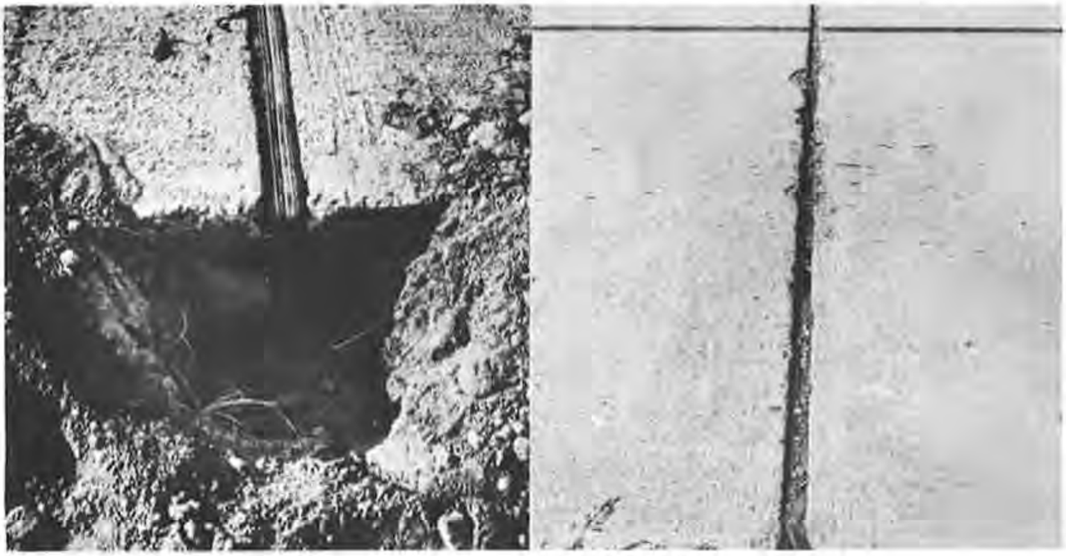


Figure 29. Air chamber expansion joint, Type TB-(4) Series 9A, Station 171+10. (Left) at time of construction, 1940. (Right) same joint 2 years later (1942) when joint had to be repaired.

TABLE 10

FAULTING OF EXPANSION JOINTS WITH AND WITHOUT LOAD TRANSFER DEVICES

Two Lanes

Series	No. of Joints Having Maximum Fault of: $\frac{1}{8}$ in.		$\frac{3}{16}$ in.		$\frac{1}{4}$ in.		Over $\frac{1}{4}$ in.		Total Joints Faulted		Total No. of Joints in Two Lanes	Percentage of Total Faulted		Load Transfer
	A	B	A	B	A	B	A	B	A	B		A	B	
10 A-1	2	1	0	0	0	0	0	0	2	1	20	10	5	$\frac{3}{4}$- by 15-in. dowels, 15 in. spacing
10 A-2	0	0	0	0	0	0	0	0	0	0	18	0	0	$\frac{3}{4}$- by 15-in. dowels, 15 in. spacing
10 B-1	5	6	5	4	0	3	0	0	10	13	18	56	72	None
10 B-2	5	5	4	1	5	5	0	2	14	13	18	78	72	None

A — Survey August 1944
B — Survey July 1949

Load Transfer Features in Expansion Joints

The merits of the different load transfer features included in the project will be evaluated on the basis of faulting and slab deflections.

Faulting. No faulting at expansion joints has developed in any series of the design project except in the case of Series 10B-1 and B-2 which include Type A-2 expansion joints with no mechanical load transfer feature. After four years in service measurable faulting occurred which has increased somewhat with age. The results of surveys made in August, 1944 and July, 1949 are summarized in Table 10. Series 10A-1 and 10A-2 were constructed with mechanical load transfer features for comparative study.

The values represent maximum faulting of slabs at the edge of the pavement only. All readings of $\frac{1}{16}$ inch and under were disregarded because of the possible influence on their accuracy of normal irregularities in the pavement surface. The data in Table 10 indicate very clearly the influence of mechanical load transfer devices on the preention of faulting at expansion joints.

TABLE 11

SUMMARY OF EXPANSION JOINT DATA RELATIVE TO SLAB DEFLECTIONS AND LOAD TRANSFER

Design Details							Slab Deflections				Load Transfer Rating[a]		
								Deflection Data, Loaded Slab			(a)	(m)	
Series	Pavement Thickness, inches	Expansion Joint Type	Load Transfer Feature	Joint Spacing, feet	Joint Filler and Seal Type	No. of Joints	Avg. Joint Opening, inches	Maximum, inches	Minimum, inches	Average, inches	Average Unloaded Slab Defl. inches	Average Relative Deflection inches	Rating of Load Transfer Unit
A 1-2-3-4	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	120	1 and 2	11	.511	.01925	.0475	.0116	.0083	.0022	43.4
B 1-2-3-4	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	240	1	15	.345	.0286	.0034	.0152	.0125	.0027	45.1
C 3-4	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	240	1	6	.325	.0327	.0090	.0191	.0179	.0019	47.5
C 1-2	9-7-9	T. E.	$1\frac{1}{4}$ x 18" Cor. Bars	240	1	9	.378	.0232	.0071	.0144	.0117	.0027	44.9
D 1-2-3-4	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	900	1 and 2	12	.317	.0434	.0020-	.0175	.0156	.0019	47.0
E 1-2-3-4	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	1800	1	9	.418	.0265	.0065	.0173	.0159	.0014	47.9
F 1-2-3-4	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	2700	1	7	.287	.0224	.0101	.0117	.0108	.0009	48.0
5 A-B-C-D	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	120	3	10	.450	.0260	.0085	.0144	.0104	.0040	41.9
6 A-B-C-D	8" Uniform	CB-1	$1\frac{1}{4}$ x 18" Cor. Bars	300	2	8	.350	.0280	.0063	.0123	.0103	.0080	45.5
7 A-B-C-D	8-6-8	DB-1	$\frac{3}{4}$ x 15" Dowel	120	2	11	.420	.0295	.0090	.0195	.0177	.0018	47.6
8 A-B-C-D	7" Uniform	CB-1	$1\frac{1}{4}$ x 18" Cor. Bars	120-300	2	7	.380	.0258	.0133	.0208	.0168	.0040	44.7
9 A	9-7-9	TB	Translode Base	100	4	3	.909	.0260	.0202	.0223	.0040	.0183	15.1
10 A1 & A2	9-7-9	DB-1	$\frac{3}{4}$ x 15" Dowel	120	5	6	.355	.0212	.0085	.0142	.0110	.0032	43.6
10 B1 & B2	9-7-9	A	Aggregate Interlock	120	2	4	.442	.0155	.0068	.0090	.0034	.0056	27.5

[a] Joint rating = $\frac{100a}{2a+m}$ Value of 50 would indicate 100 percent load transfer ability. Measurements made in summer and fall of 1948.

No comparative faulting data are available in regard to the relative merits of the remaining types of expansion joint designs which were included in the load transfer and joint design study. Evidently time and traffic have not been sufficient to bring out any noticeable physical differences as yet.

Slab Deflections. During the summer and fall of 1948 a series of slab deflection measurements were made at certain expansion joints in an attempt to evaluate the load transfer characteristics of the different types of units included for study. In all cases the axle load employed was 18,000 pounds supported on two single wheels. The outside wheel was placed 6 inches from the pavement edge. The load was transferred alternately from one slab corner to the other. Total and relative deflections were measured by one-thousandth dials attached to supports on the shoulder. All readings were taken in the morning of each day. Three separate observations were made at each joint per day. A summary of load deflection data taken at two seasons of the year, summer and fall, have been averaged to give final results, which are presented in Table 11.

The data in Table 11 brings out some very interesting and significant points. (1) With reference to Series 10, Section 10B-1 and B-2 without load transfer has a joint rating of only 27.5 compared to 43.6 for Section 10A-1 and A-2 with load transfer. Apparently in Series 10B-1 and B-2 the joint filler under pressure is developing a certain amount of mechanical interlock between the joint faces. (2) The Type TB (4) expansion joints in Series 9A with translode base units have the lowest load transfer rating of 15.1. Comparable expansion joint design in Series 5 with $\frac{3}{4}$-inch dowels has a rating of 41.9. Apparently the translode base unit is not a satisfactory load transfer device. (3) At present no definite distinction can be drawn between the other types of expansion joint design since they all have approximately the same joint rating. These readings range from 43.4 to 48.0 which is indicative of good load transfer performance to date.

CONTRACTION JOINT DESIGN

The types of contraction joint design considered in the investigation are:

Type DB	Department Standard consisting of $\frac{3}{4}$- by 15-inch dowels at 15-inch spacing with $2\frac{1}{2}$- by $\frac{1}{4}$-inch premolded fiber filler strip at top.
Type 1B	Same as above except that a $\frac{1}{2}$- by $2\frac{1}{2}$-inch groove is substituted for the premolded fiber filler strip.
Type 2A	Same as Type DB except for the addition of a 1-inch high metal parting strip at bottom of joint.
Type 2B	Same as Type 2A except the groove was substituted for the premolded fiber filler strip.
Type 3	Metal divider plate full depth and groove at top. $\frac{3}{4}$- by 15-inch dowels at 15-inch spacing used.
Type 4	Continuous plate dowel assembly. Top edged and sealed with asphalt-rubber joint compound.

Type 5	Keylode plate dowel assembly. Not sealed.
Type CB	$1\frac{1}{4}$-inch by 18-inch corner bars with premolded fiber filler strip, placed 9 inches from edges.
Type 6	Aggregate interlock only.

At the end of 10 years sufficient evidence has been collected on the relative behavior of the various types of contraction joint construction to warrant detailed discussion.

Premolded Fiber Strip versus Sealed Groove

Types DB and 1B were constructed primarily to study the possibility of reducing spalling at contraction joints by substituting the groove for the premolded fiber filler strip. In that respect the following relative behavior has been observed:

Spalling at Transverse Edges. In Series 5 the majority of the contraction joints were constructed with a groove which was subsequently filled with a bituminous-latex sealing compound. At the present time the joints are in excellent condition except for weathering of the seal as may be seen in Figure 30. The joint edges have remained intact and no spalling of the concrete has been noted. Some scaling has appeared at joint edges. In the case of contraction joints constructed with premolded fiber strip several kinds of failure have been encountered, the most undesirable of which is spalling of the concrete along the joint edge. For comparable data on joint spalling see Table 12, also, Tables 7 and 8. Perhaps the spalling is a direct outcome of installation practice since it is usually associated with tipping of the fiber strip during installation or placing the strip too far below the surface of the pavement. Typical examples of this kind of spalling may be noted in Figure 31. The spalling of the joints in Sections F and G of Series 5 is definitely associated with the type of load transfer device rather than type of joint construction. This will be discussed later under Continuous Plate Dowel Construction.

Spalling at Longitudinal Joints. Corner spalling at junction of transverse and longitudinal joints is becoming quite prevalent throughout the Design Project wherever bituminous premolded fiber strip is used in the construction of both type of joints. This

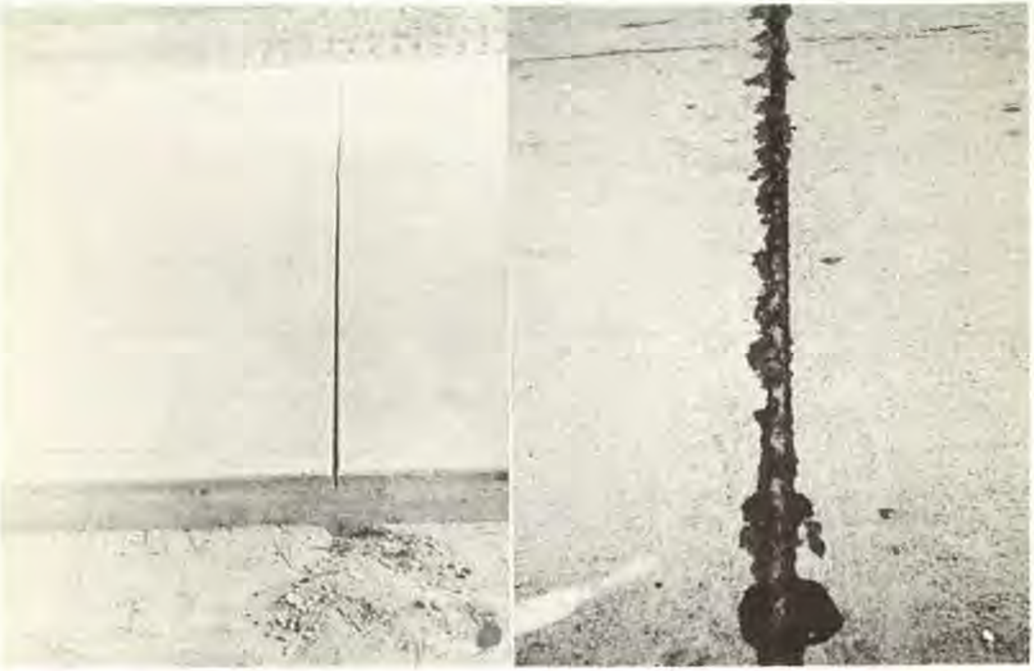


Figure 30. Type 1B contraction joint with groove, Station 64+90. (Left) joint when installed, 1940. (Right) same, joint, 1950.

TABLE 12

SUMMARY OF PAVEMENT CRACKING AND SPALLING IN RELATION TO CONTRACTION JOINT DESIGN

		Series 5 9"-7"-9" Slabs			30 Foot Slabs 37# Steel - 120" Expansion Joints					
					Cracking in feet				Design	
Section No.	Joint Design Type	Total Slabs	Number Cracks	Percent Cracks	Transverse	Diagonal	Longitudinal	Total	Construction	Load Transfer
A	1B	12	0	0	0	0	0	0	Premolded fiber strip	3/4-in. Dowel
B	2A	12	0	0	0	0	0	0	Fiber strip &Parting Strip	3/4-in. Dowel
C	2B	12	0	0	0	0	0	0	Groove	3/4-in. Dowel
D	3	12	0	0	0	0	0	0	Groove & metal plate	3/4-in. Dowel
E	3	12	0	0	0	0	0	0	Same as D	3/4-in. Dowel
F	4	12	1	8	11	0	0	11	Edged & sealed	Continuous Metal plate
G	4	6	2	33	50	0	0	50	Edged & sealed	Same

SPALLING

		Number Joints	Number Spalled	Percent Spalled	Number Spalled Areas	Percent for Group	Joint Design Method of Seal
A	1B	12	1	8	1	17	Premolded Bituminous Fiber Strip
B	2A	12	3	25	4		
C	2B	12	0	0	0		
D	3	12	0	0	0	0	Grooved and Sealed
E	3	12	0	0			
F	4	12	7	58	10	50	Grooved and sealed surface Spalling due to load transfer device
G	4	6	2	33	2		

particular type of spalling has not developed at present in Series 5C and D where the grooved transverse joints were constructed. Typical examples of this type of spalling are illustrated in Figure 32.

Sealing of Joint. Another common fault of the premolded fiber strip is that it does not provide adequate seal, especially during the winter months when the joints are at their maximum opening. In this condition the filler strip is loose in the joint, thus permitting the infiltration of water and inert material. Typical examples of this condition are presented in Figure 33. The seriousness of this condition is naturally affected by joint spacing, being greater in the sections with 60-foot joint spacing than in the sections with 10-foot spacing. In many instances it was noted that at the end of 10

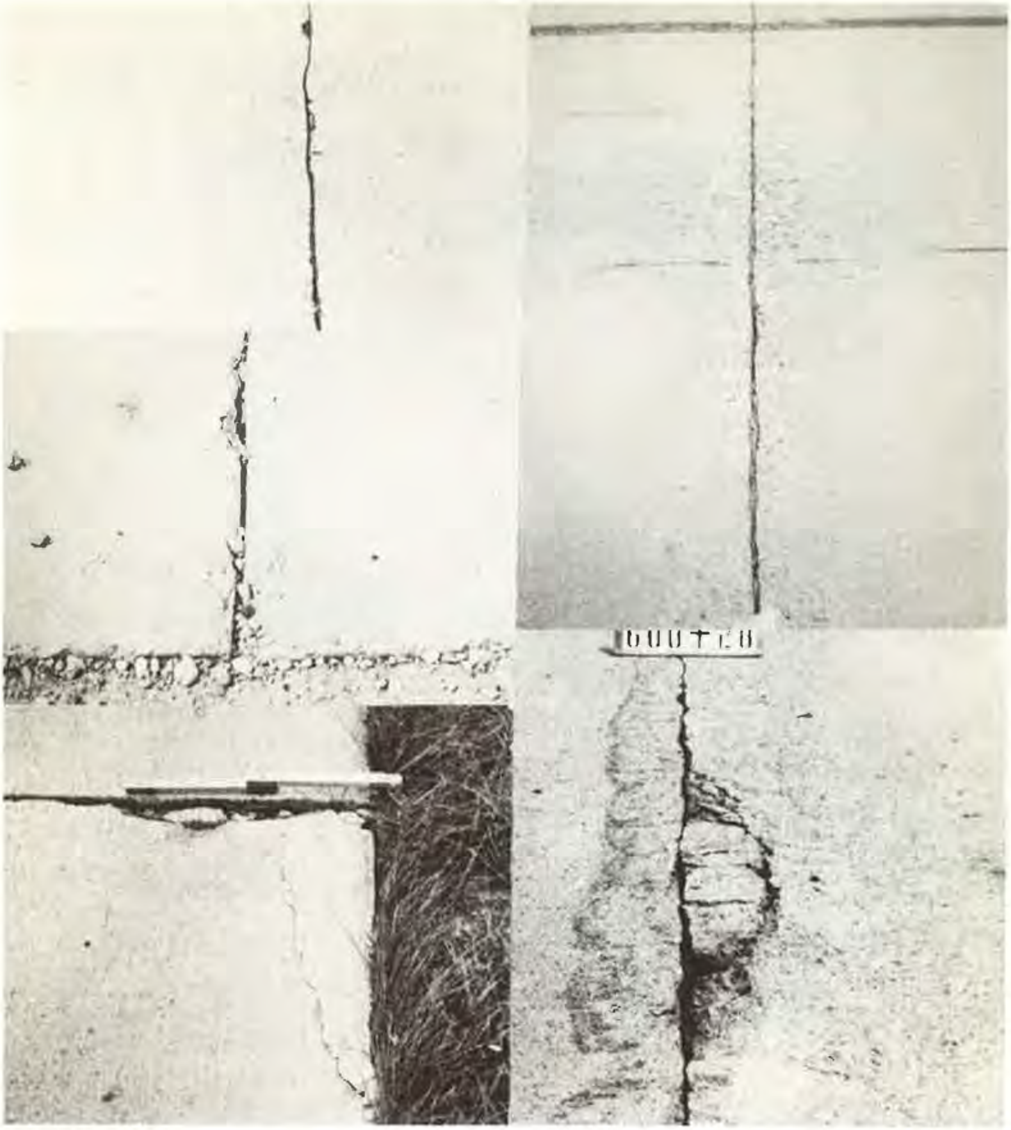


Figure 31. Typical failures common to contraction joints constructed with premolded fiber strips.

years the filler strip has become rotten and displaced by traffic. This may be observed in Figure 31.

Load Transfer Features

Several types of load transfer features in contraction joints were considered; standard $\frac{3}{4}$- by 15-inch dowel bars at 15-inch spacing, aggregate interlock, special corner bars and the continuous plate dowel. These features will be discussed in the order mentioned.

Standard $\frac{3}{4}$-Inch Dowel. The standard $\frac{3}{4}$-inch dowel assembly was used throughout the Design Project with the exception of those series in which special load transfer features were incorporated for comparative study. The performance of these units will be discussed later on in conjunction with other factors associated with joint design.

Dowel Bars versus Aggregate Interlock. In this study two major factors were con-

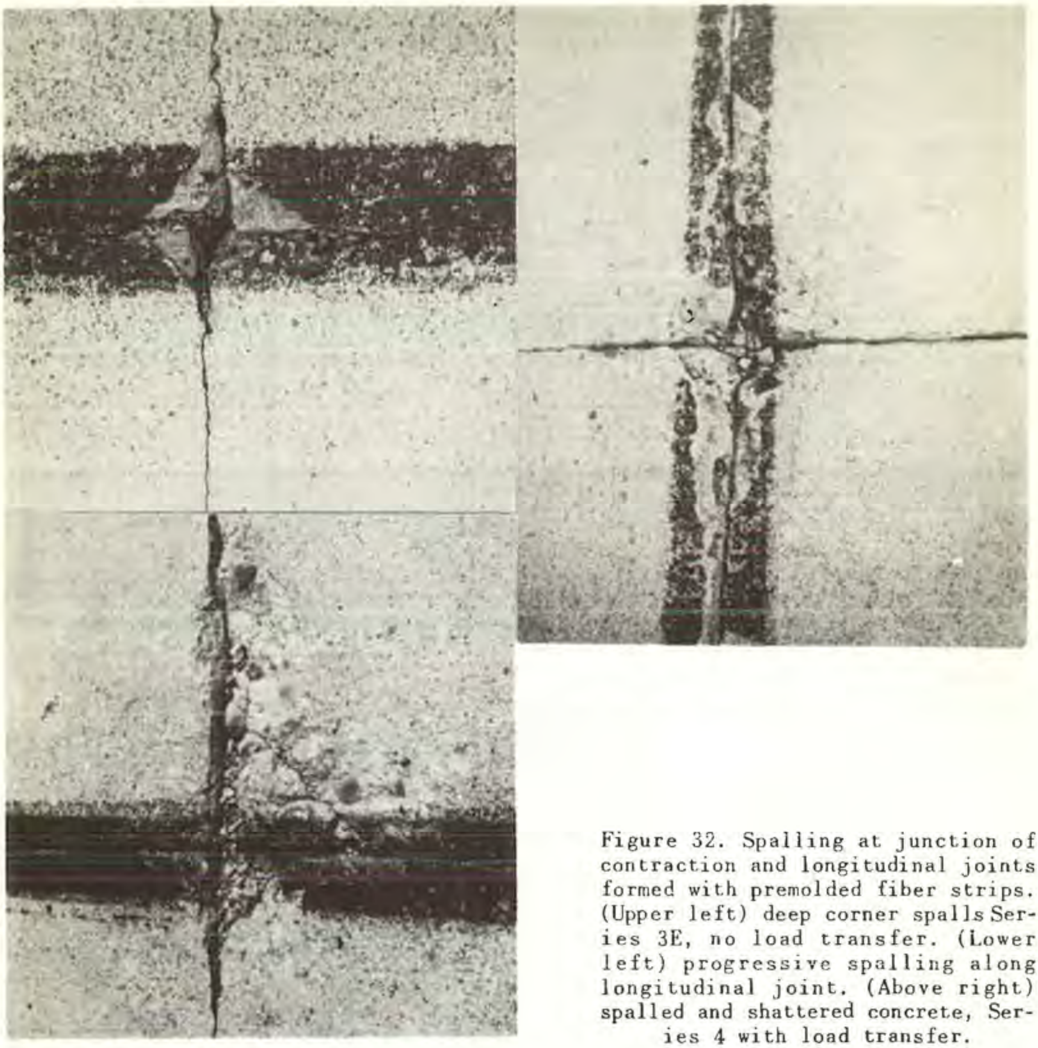


Figure 32. Spalling at junction of contraction and longitudinal joints formed with premolded fiber strips. (Upper left) deep corner spalls Series 3E, no load transfer. (Lower left) progressive spalling along longitudinal joint. (Above right) spalled and shattered concrete, Series 4 with load transfer.

sidered: (1) the omission of dowels in contraction joints with normal expansion joint spacings of 120 feet which would naturally permit the intermediate contraction joints to open freely; and (2) similar construction with the spacing of expansion joints at great distances which would effect considerable restraint on individual slab movement and thus develop a better joint condition for aggregate interlock to perform in the manner intended.

In Series 10A and 10B contraction joints were constructed with and without dowel bars at joint spacings of 15 and 20 feet, the expansion joint spacing remaining constant at 120 feet in all sections of the series. Results of surveys conducted in the summers of 1944 and 1949 are presented in Table 13. The data in Table 13 indicate that the number of faulted joints in the undoweled sections of Series 10 is considerably greater than that in the doweled sections. In all cases the degree of faulting represented indicates the amount that the corner of the approach slab was below that of the passing slab. Moreover, there is evidence that differential movement of the slabs has started at the longitudinal joints due to the absence of dowels across the transverse joint. Furthermore, the fact that a considerable number of joints with dowel bars show faulting would indicate the inadequacy of the $^3/_4$-inch dowel at such a spacing of 15 inches. The study clearly indicates the beneficial effect of dowels in contraction joint construction, especially when expansion joints are provided at distances of 120 feet as in this case.

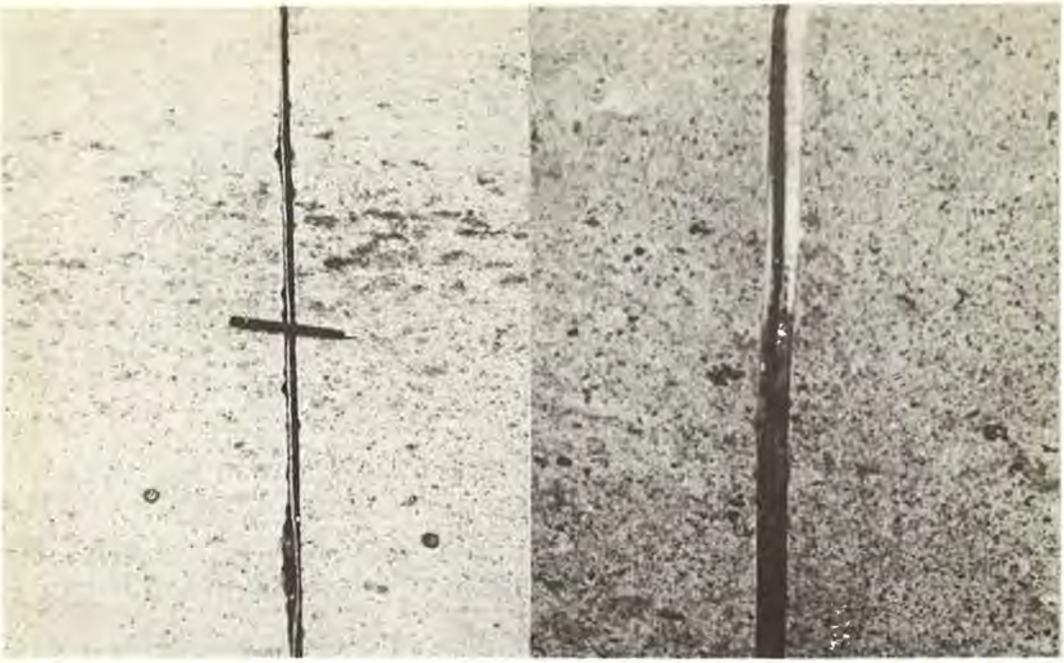


Figure 33. Typical condition of contraction joints in winter with bituminous premolded fiber strip, Series 1A, 60-ft. contraction joint spacing. Winter 1944-45. (Left) open to permit intrusion of inert material. (Right) snow and ice forms in joint.

TABLE 13

FAULTING OF CONTRACTION JOINTS WITH AND WITHOUT LOAD TRANSFER DEVICES

	Two Lanes													
	Number of Joints Having Maximum Fault of:								Total Joints Faulted		Total No. of Joints in Two Lanes	Percentage of Total Faulted		
	$\frac{1}{8}$ in.		$\frac{3}{16}$ in.		$\frac{1}{4}$ in.		Over $\frac{1}{4}$ in.							
Series	A	B	A	B	A	B	A	B	A	B		A	B	Load Transfer
10 A-1	7	9	0	1	1	1	0	2	8	13	90	9	15	$\frac{3}{4}$- by 15-in.
10 A-2	4	6	1	3	0	0	0	1	5	10	126	4	8	dowels, 15-in. spacing
10 B-1	23	26	7	11	4	9	0	0	34	46	90	38	51	No dowels,
10 B-2	22	37	5	4	6	12	0	1	33	54	126	26	43	agg. interlock
3D	0	5	0	0	0	0	0	0	0	5	176	0	3	$\frac{3}{4}$- by 15-in.
4D	1	5	0	0	0	0	0	1	1	6	178	1	3	dowels, 15-in. spacing
3E	15	32	0	7	1	1	0	0	16	40	178	9	22	No dowels,
4E	3	9	0	0	0	0	0	0	3	9	358	1	3	agg. interlock

A – Survey August 1944
B – Survey July 1949

In Series 3E and 4E dowel bars were omitted at the contraction joints for the purpose of studying the effect of slab movement restraint on the performance of aggregate interlock. The expansion joint spacing in both cases is 1, 800 feet and contraction joint spacings are 20 feet and 10 feet, respectively. In conjunction with Series 3E and 4E, Series 3D and 4D, with the same contraction joint spacing as Series 3E and 4E, re-

TABLE 14

SUMMARY OF CONTRACTION JOINT DATA RELATIVE TO SLAB DEFLECTIONS AND LOAD TRANSFER

Design Details						Slab Deflections				Load Transfer Rating[a]		
							Deflection Data, Loaded Slab			(a)	(m)	
Series	Pavement Thickness, inches	Expansion Joint Spacing	Contraction Joint Type	Load Transfer	No. of Joints	Avg. Joint Opening, inches	Maximum, inches	Minimum, inches	Average, inches	Average Unloaded Slab Defl. inches	Average Relative Deflection inches	Load Transfer Joint Rating
A 1-2-3-4	9-7-9	120	DB	¾" x 15" Dowels	15	.193	.0120	.0050	.0072	.0052	.0020	41.9
B 1-2-3-4	9-7-9	240	DB	¾" x 15" Dowels	12	.162	.0158	.0052	.0127	.0110	.0017	46.4
C 1-2-3-4	9-7-9	240	DB	¾" x 15" Dowels	11	.060	.0185	.0030	.0096	.0082	.0014	46.0
D 1-2-3-4	9-7-9	900	DB	¾" x 15" Dowels	11	.095	.0503	.0100	.0228	.0212	.0016	48.2
E 1-2	9-7-9	1800	DB	¾" x 15" Dowels	5	.068	.0325	.0198	.0273	.0225	.0048	45.2
E 3-4	9-7-9	1800	6	Aggregate Interlock	12	.029	.0130	.0045	.0078	.0062	.0016	44.3
F 1-2-3-4	9-7-9	2700	DB	¾" x 15" Dowels	17	.024	.0825	.0010	.0058	.0049	.0009	45.8
5 A-B-C	9-7-9	120	1B-2A-2B	¾" x 15" Dowels	10	.239	.0219	.0077	.0139	.0105	.0034	43.0
5 D-E	9-7-9	120	3	¾" x 15" Dowels	6	.204	.0150	.0055	.0101	.0086	.0015	46.0
5 F	9-7-9	120	4	Continuous Plate Dowel	3	.252	.0070	.0035	.0057	.0053	.0004	48.2
6 A-B-C-D	8" Uniform	120-300	CB	1¼" x 18" Corner Bars	5	.100	.0327	.0022	.0192	.0132	.0060	40.8
7 A-B-C-D	8-6-8	120	DB	¾" x 15" Dowels	8	.161	.0332	.0045	.0211	.0175	.0036	45.3
8 A-B-C-D	7" Uniform	120-300	CB	1¼" x 18" Corner Bars	12	.080	.0355	.0080	.0175	.0146	.0029	45.5
9 T S	9-7-9	180	5	Keylode	3	-	.0118	.0060	.0082	.0055	.0027	40.2
10 A1 A2	9 7 9	120	DB	¾" x 15" Dowels	3	.133	.0208	.0108	.0154	.0114	.0040	42.8
10 B1 - B2	9-7-9	120	6	Aggregate Interlock	5	.172	.0155	.0035	.0116	.0009	.0107	7.2

[a] Joint rating $\frac{100\,a}{2\,a + m}$ A rating of 50 equals 100 percent load transfer. Measurements taken in summer and fall of 1948.

spectively, but containing dowel bars, were chosen for comparative study. In this case the expansion joint spacing is 900 feet instead of 1,800 feet. Results of surveys made in 1944 and 1949 are also shown in Table 13.

The data show in all cases a gradual increase in faulting with time. Faulting of the 20-foot slabs in Series 10 with no slab restraint is considerably higher than that in Series 3E with some slab restraint. However, even in the latter case, it is clearly indicated that aggregate interlock is not entriely effective in preventing faulting. Further, the data for Series 4D and 4E indicate that aggregate interlock is beginning to lose its effectiveness in the more restrained sections also.

The absence of dowels has created a weakness in the pavement structure at the junction of longitudinal and transverse joints which eventually may give rise to a serious maintenance problem. This weakness is manifested by noticeable differential movement of the slab corners at the intersection of the transverse joints and the longitudinal joint which has resulted in spalling of the concrete at the joint intersection and is slowly progressing along the longitudinal joint. However, since 1945 spalling at the same location in doweled joints has occurred to a considerable extent where premolded joint seal was used. Figure 32, presented earlier, shows typical contraction and longitudinal joint conditions with and without dowels.

Corner Bars. In Series 6 and 8, 1¼- by 18-inch corner bars, placed 9 inches in from the slab edges were substituted for the standard ¾-inch dowel bar assemblies at all contraction joints. At the present time there is no discernible physical condition of the joints which would indicate that they are not performing in a satisfactory manner.

Load deflection data presented in Table 14 indicates that the average of the load transfer ratings for Series 6 and 8, is 43.1 percent. This value is approximately the same as that of 43 obtained by averaging the load deflection results of joints in A sections of Series 1-2-3 and 4 and Sections A, B and C of Series 5, as well as all sections in Series 7. This would indicate that corner bars under certain local conditions may have merit in transferring load across a joint as compared to a system of dowels. Furthermore, a comparison of joint rating values for Series 6 and Series 8 will show that Series 6 has a lower joint rating than Series 8, in spite of the fact that Series 6 has a thicker slab. It is believed that this difference in joint performance is due primarily to foundation conditions, rather than joint design. The relative subgrade conditions under the two test sections has been fully explained previously under Pavement Performance in Relation to Cross Section.

Continuous Plate Dowel. Two types of continuous plate dowel assemblies in common use at the time were considered for comparative study. One particular unit, designated at Type 5 (Keylode) and employing aggregate interlock in conjunction with a plate dowel was installed at three contraction joints in Section 9-TS between Stations 180+10 and 181+90. The other plate dowel unit known as Type 4 was installed at all contraction joints in Series 5, Sections F and G.

Views illustrating typical conditions of Type 5 joint at construction and 10 years later are presented in Figure 34. In Figure 34, two physical weaknesses are in evidence.

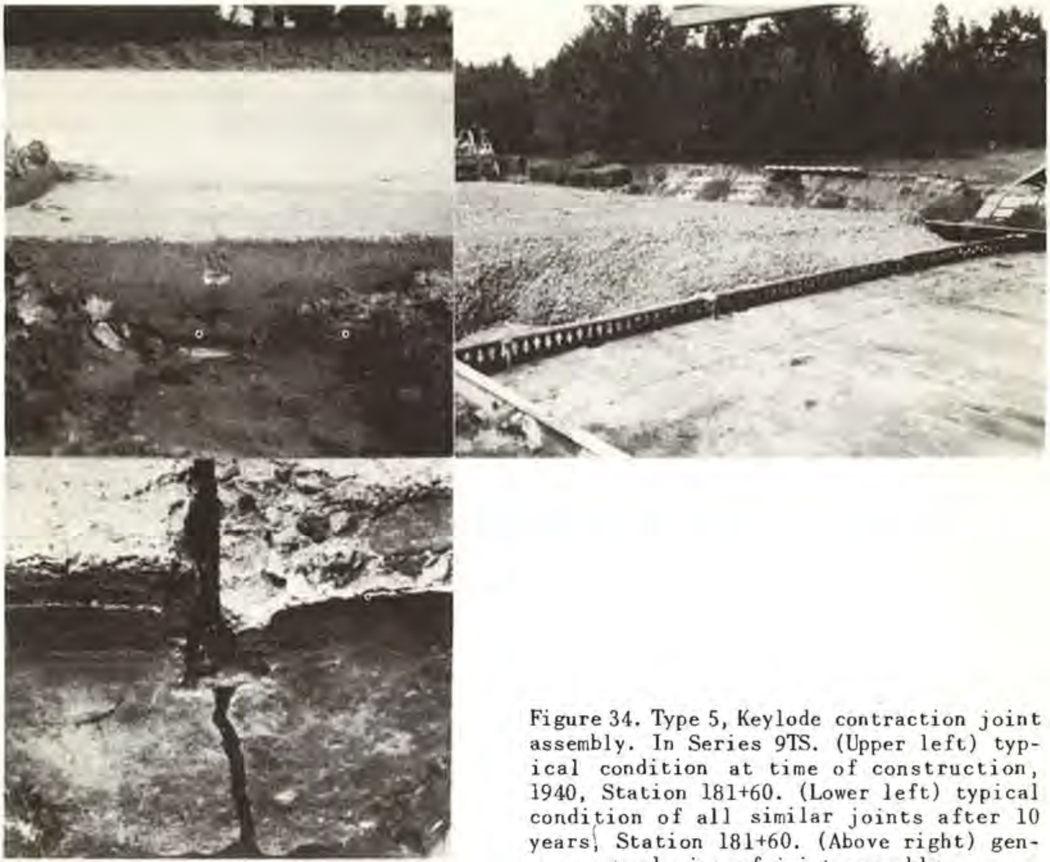


Figure 34. Type 5, Keylode contraction joint assembly. In Series 9TS. (Upper left) typical condition at time of construction, 1940, Station 181+60. (Lower left) typical condition of all similar joints after 10 years, Station 181+60. (Above right) general view of joint assembly.

In the first place, the joint assembly provides no seal at top or sides against the infiltration of water or inert material. Note how the upper portion of the joint opening is full of inert particles. Second, it is apparent upon examination that the projections of concrete which extended to form the aggregate interlock are broken, thus destroying any load transfer action from that source.

Load deflection studies on these joints indicate a very good load transfer rating (see Table 14). Values comparable to standard doweled joints were obtained.

The continuous plate dowel, Type 4 located in Series 5, has an inherent design weakness which causes spalling along the joint edge. Examples of the type of spalling may be seen in Figure 35. An examination of the joint revealed that the plates invariably were frozen in place due to rust and consequently the joints were not functioning as designed. The diagrammatic sketch in Figure 36 shows the general condition of the pavement in Series 5 and especially the degree of spalling in Sections F and G caused by the plate dowel assemblies.

As may be observed in Table 14, the joint rating of 48.2 indicates excellent load transfer properties for this type of joint unit.

Crack Control Methods

The examination of contraction joints in many concrete projects as well as those in the Michigan Test Road revealed that the spontaneous cracking of the pavement at the plane of weakness joints was very irregular and in general not vertical as desired. Also it was observed that in many cases diagonal cracking and subsequent spalling was common at the bottom of the contraction joints.

With these facts in mind, two well-known devices for controlling cracking of the pavement at contraction joints were installed in Series 5 for comparative study, namely parting strips in bottom of joints and metal divider plates.

Parting Strips. In this case the parting strips consisted of one-inch wide No. 19 gage metal strip fastened in a vertical position to the bottom of the joint assemblies, directly under the premolded fiber strip or the groove, whichever was used to form the plane of weakness. These joints are designated Types 2A and 2B. They were installed in Series 5B and 5C.

In order to attain proper results with the metal parting strip placed at the bottom of the joint careful workmanship must be exercised. Methods must be employed that will insure the proper placing of the metal parting strip directly under the premolded filler strip or groove, otherwise undesirable cracking will result. When properly placed, vertical cracking of the pavement will take place (see Figure 37).

Steel Divider Plates. This type of joint (Type 3) was installed in Series 5D and 5E. Construction consisted of a vertical 22-gage continuous metal plate extending the full depth of the pavement to break the continuity of the concrete. In this case, the metal dividing plate was used with the groove.

There is no question that the full continuous metal plate will insure positive cracking of the joint.

Effect of Crack Control Methods on Load Transfer. With reference to Table 14 the data show that the joints with metal divider plates have a lower load transfer rating than those of normal construction. This is to be expected since no aggregate interlock is involved.

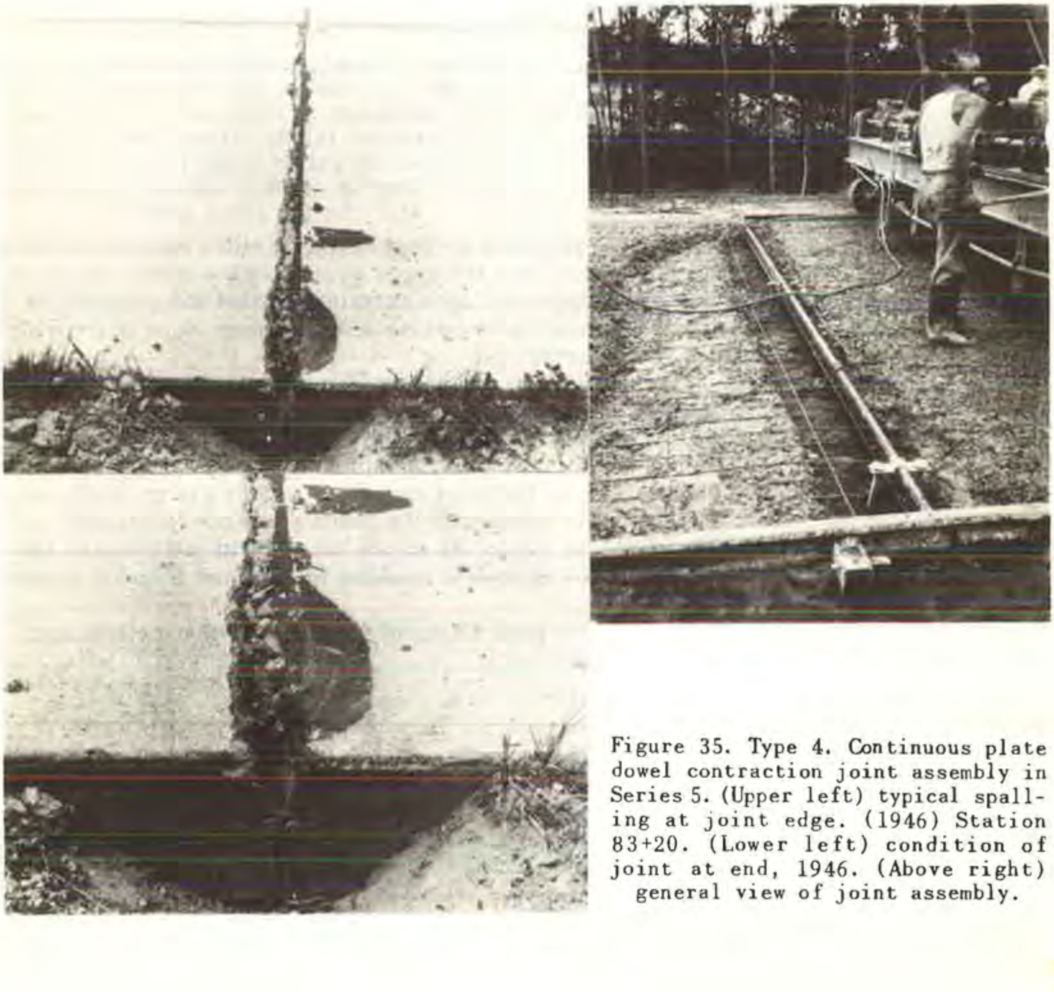


Figure 35. Type 4. Continuous plate dowel contraction joint assembly in Series 5. (Upper left) typical spalling at joint edge. (1946) Station 83+20. (Lower left) condition of joint at end, 1946. (Above right) general view of joint assembly.

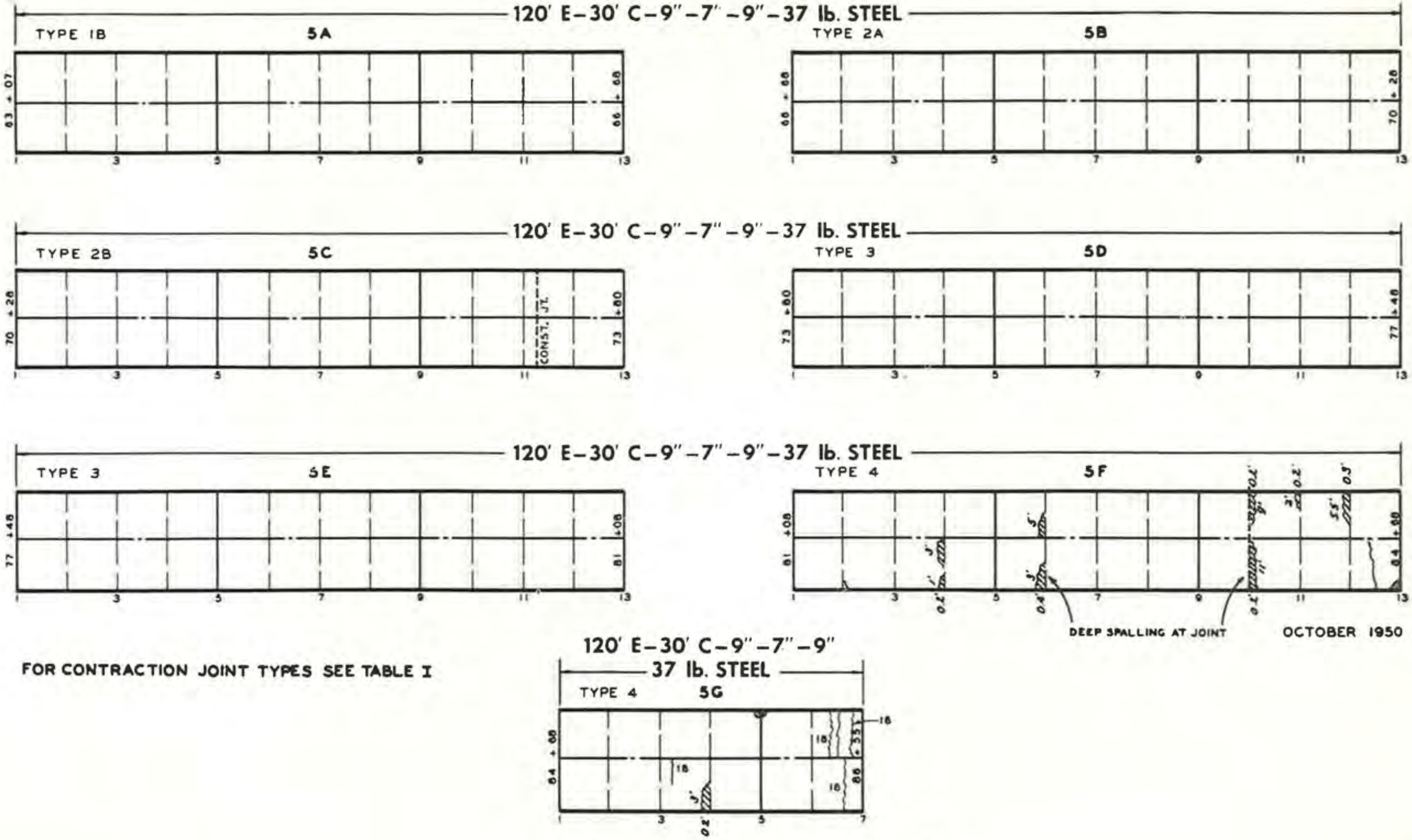

120' E-30' C-9"-7"-9"-37 lb. STEEL
TYPE 1B
5A
63 + 07
66 + 68
TYPE 2A
5B
66 + 68
70 + 28
120' E-30' C-9"-7"-9"-37 lb. STEEL
TYPE 2B
5C
70 + 28
CONST. JT.
73 + 80
TYPE 3
5D
73 + 80
77 + 48
120' E-30' C-9"-7"-9"-37 lb. STEEL
TYPE 3
5E
77 + 48
81 + 08
TYPE 4
5F
81 + 08
84 + 68
DEEP SPALLING AT JOINT
OCTOBER 1950
120' E-30' C-9"-7"-9"
37 lb. STEEL
TYPE 4
5G
84 + 68
86 + 35
18
FOR CONTRACTION JOINT TYPES SEE TABLE I
1
3
5
7
9
11
13

Figure 36. Condition of pavement in Series 5.

Bituminous-Rubber Joint-Sealing Compounds

Three types of bituminous-rubber joint-sealing compounds were used on the Design Project for comparative study in conjunction with joint design. These materials consisted of two types of asphaltic oil-latex compounds developed by the department and a commercial type of hot-poured rubber-asphalt compound known as Thermoplastic No. 52144.

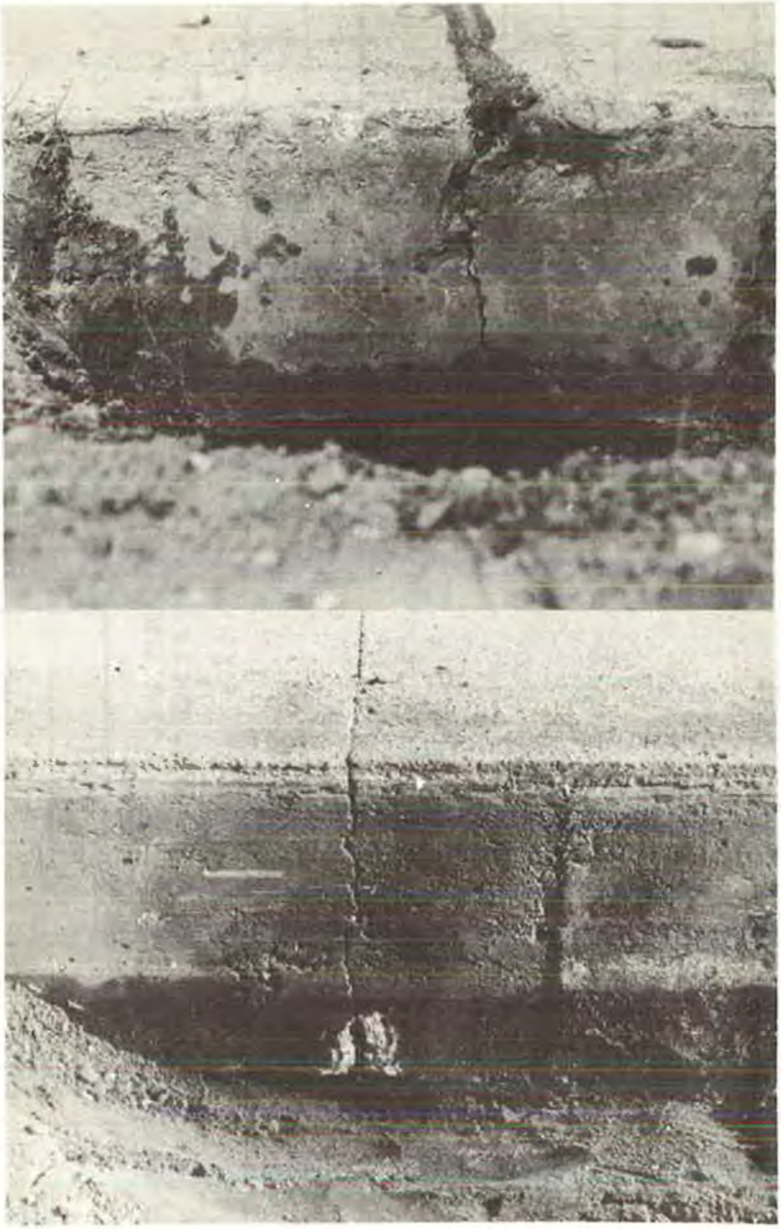


Figure 37. Effect of bottom parting strip on joint cracking. (Top) typical cracking at plane of weakness joint without parting strip at bottom. (Bottom) typical cracking when parting strip is properly installed.

One type of asphalt-latex compound, designated Type 1, consisted of a mixture of

TABLE 15
SUMMARY OF JOINT SEAL DATA

Joint Conditions [a]	Evaluation in Percent					
	Asphalt-Latex, Type 1		Asphalt-Vultex, Type 2		Thermoplastic, Type 5	
	1945	1949	1945	1949	1945	1949
Effective seal against water	30	0	25	0	33	33
Effective seal against dirt	33	30	33	30	33	33
Plasticity	18	5	20	5	31	16
Overall rating	81	35	78	35	97	82

[a] All three items used in evaluation have equal weight, so that a rating of 33 percent for a given item indicates perfect condition for that item.

70 parts of asphaltic oil SC-6A, 30 parts of normal rubber latex and 2 parts of hydrated lime. The materials were mixed together under controlled conditions immediately prior to sealing the joint. The other type, designated Type 2, was identical with Type 1, except a commercial vulcanized latex known as Vultex was substituted for the normal rubber latex.

The Thermoplastic compound, designated as Type 5, was a hot-poured type commercial rubber compound furnished in block form which upon heating to 450 deg. F transforms to a liquid of proper consistency for pouring into prepared joints.

These materials were installed during the construction of the project as part of the contract. Their respective locations are given in Table 1.

During 10 years of service, none of the sealing compounds has required maintenance at any time. The materials have weathered in varying degrees, however, and have become considerably more inspissated with age.

Condition surveys made in 1945 and 1949 revealed a measurable difference in service behavior of the three products as shown in Table 15. The condition rating values given in Table 15 are based on the apparent plasticity and effectiveness of seal against water and dirt as manifested by the degree of cohesion and bond failure evident in each joint.

The asphaltic oil-latex compounds have reverted to a putty-like consistency, losing all their original plasticity. Permanent cracks have formed in the materials (see Figure 38).

The Thermoplastic hot-pour rubber-type compound is still in excellent condition after 10 years of service (see Figure 38).

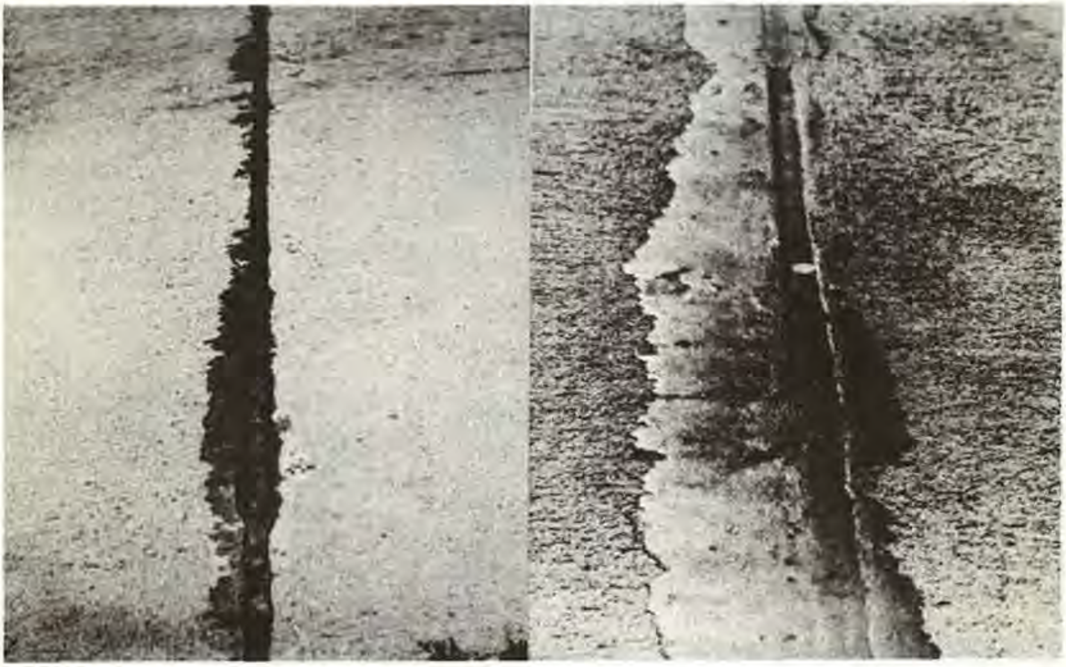


Figure 38. Present condition of bituminous-rubber material after 10 years. (Left) asphalt-latex sealing compound. (Right) thermoplastic sealing compound.

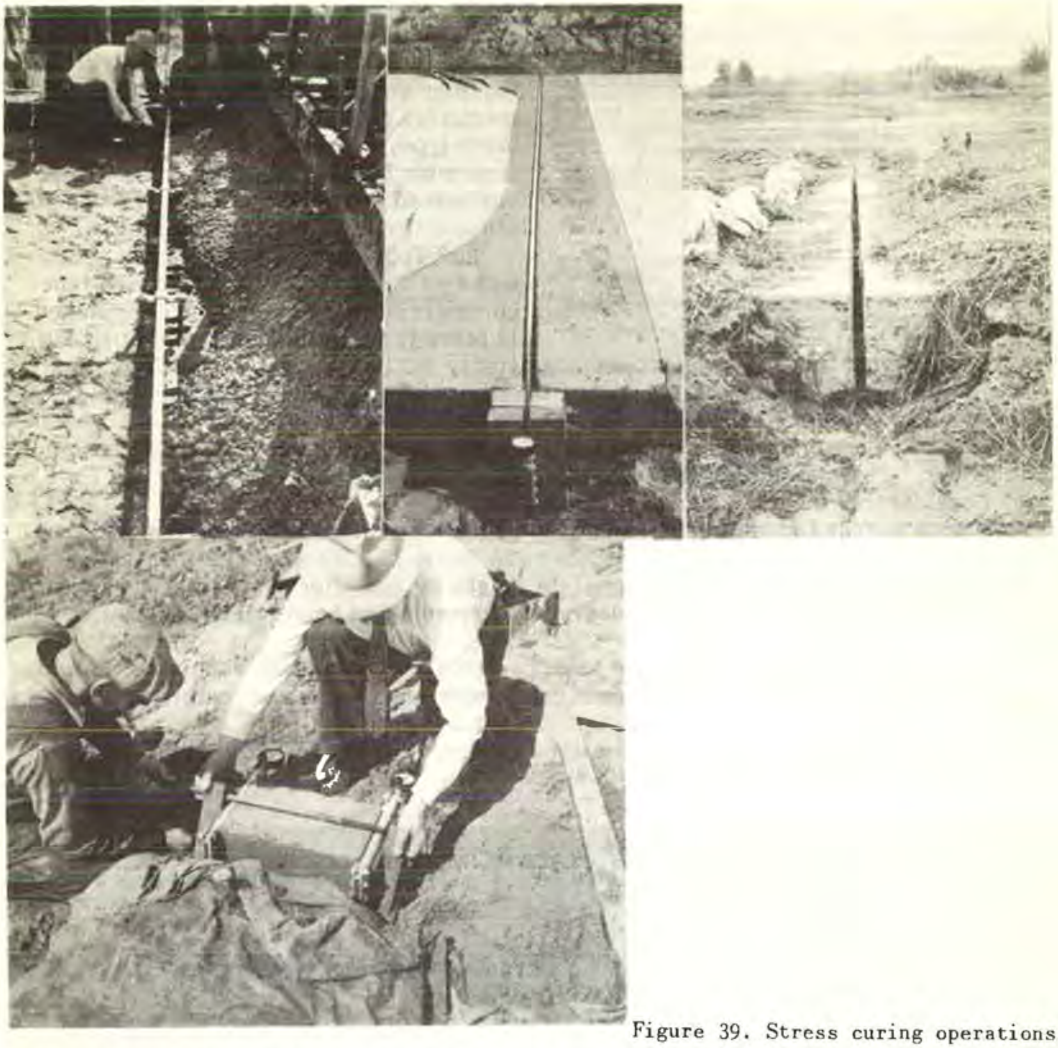


Figure 39. Stress curing operations.

The study definitely indicates that the new rubber-type joint seals are superior to the straight asphalt or tar products commonly used for sealing joints and cracks in pavement.

Summary

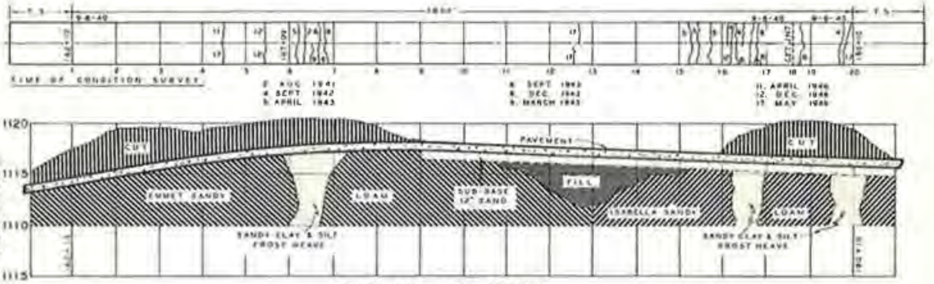
In review, the data presented in connection with the design of transverse joints disclose several significant facts. They are: (1) from the standpoint of both construction and performance the non-extruding premolded fiber board expansion joint has many more desirable features than the air chamber type of expansion joint used on the project; (2) due to the large seasonal movement of the contraction joints, aggregate interlock in itself is not sufficient to provide adequate load transfer to prevent faulting of slabs; (3) the practice of providing ¾-inch dowels at 15-inch spacings is not adequate to prevent faulting of slabs either at expansion or contraction joints; (4) certain types of plate dowel load transfer units are to be avoided; (5) the premolded fiber strip used in forming contraction and dummy joints is a detriment to good concrete pavement construction. Unless great care is exercised both in the placing of the strip and during subsequent finishing operations, irreparable damage results to the pavement. (6) Bituminous rubber type joint-sealing compounds are superior to straight asphalt or tar products commonly used in sealing joints.

Incidental Studies

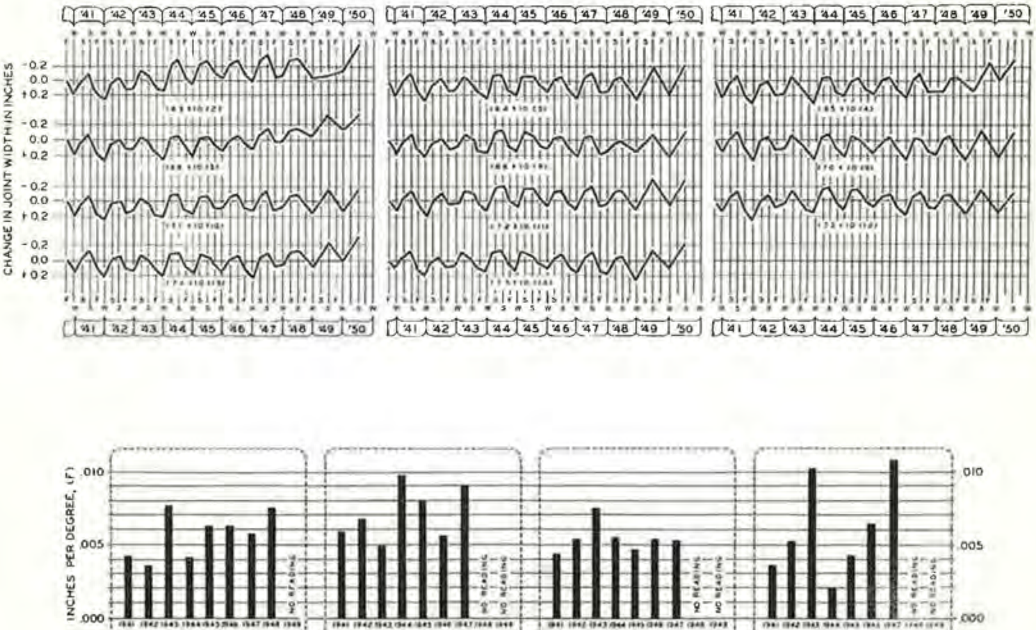
In addition to the major investigations embodied in the Michigan Test Road, several incidental studies were introduced into the program. These studies pertained primarily to various construction methods. The results of two of these studies, namely, stress curing of concrete and vertical behavior of the pavement are of enough importance to warrant their inclusion in this report.

STRESS CURING OF CONCRETE

In Series 9A, 1, 800 feet of concrete pavement was placed by the stress curing method, which eliminates reinforcement and transverse joints other than those provided for expansion. The slabs were laid in 100-foot lengths. The concrete was subjected to controlled compressive forces during the 7-day curing period, or until such time as


TIME OF CONDITION SURVEY
1120
1115
1110
ELEVATION
CUT
PAVEMENT
FILL
EMMET SANDY
LOAM
SUB-BASE 12" SAND
ISABELLA SANDY
SANDY CLAY & SILT FROST HEAVE

PLAN AND SOIL PROFILE



DAILY CHANGE IN EXPANSION JOINT WIDTH

Figure 40. Stress cured concrete.

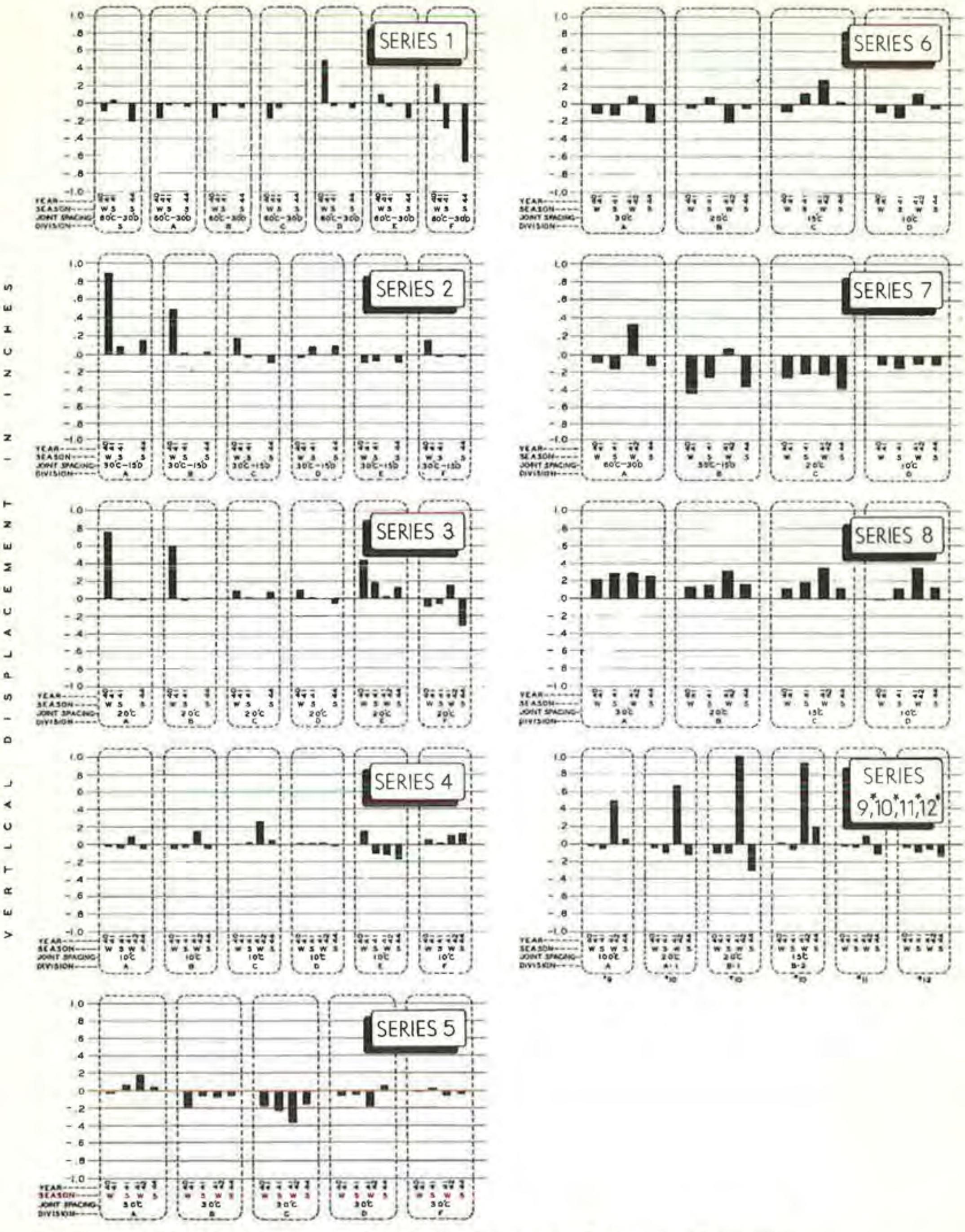


Figure 41. Average vertical displacement of pavement by series.

the beams for modulus of rupture tests reached the 7-day specification strength of 550 psi. The application of pressure was accomplished by using canvas covered rubber hose pressure cells inserted in the expansion joint openings. The pressures were increased at a rate controlled by determinations of strength increase in test specimens up to a maximum of 200 psi. (see Figure 39).

Physical Condition of Slabs

At the end of 10 years, 10 of the 18 slabs are in apparently perfect condition. The remaining 8 slabs have cracked as shown in Figure 40. The first crack in the entire test section occurred in the slab between Joints 6 and 7 prior to the survey conducted in August, 1941. The progressive development of cracks is also illustrated in Figure 40 by the numbers appearing at each crack.

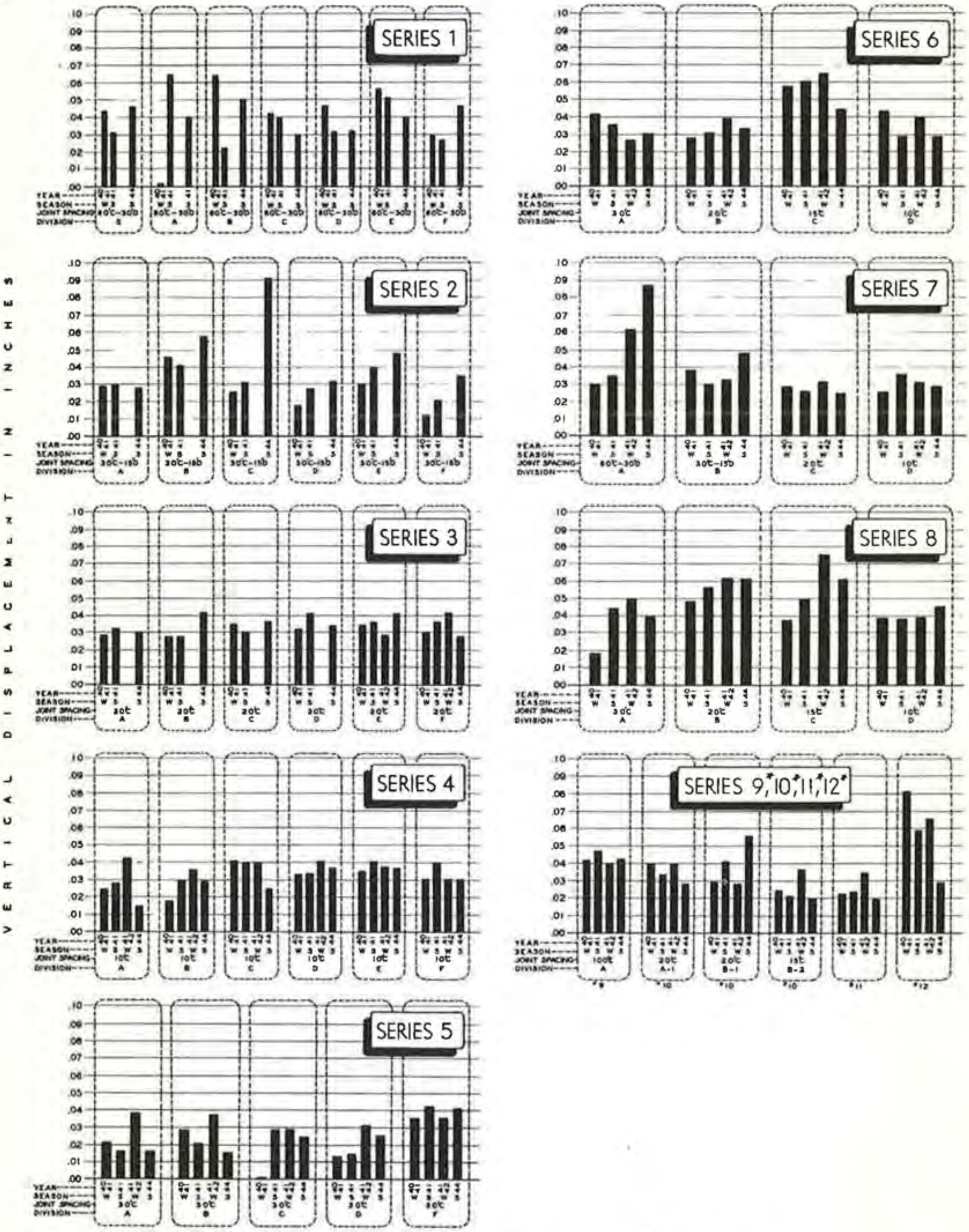


Figure 42. Average relative displacement of slab ends.

A careful analysis of the cracked slabs and subgrade has definitely proven that the cracking in four of the slabs can be attributed directly to abnormal changes in the subgrade caused by undesirable soil conditions and not to any factor or weakness in the slab structure due to method of construction. Part of Series 9A was constructed on a 12-inch sand subbase overlying a loamy sand with clay pockets and sandy clay loam soils. Apparently the 12-inch sand subbase was not thick enough, since there is evidence of rutting and intermingling of the subbase and subgrade materials causing improper drainage under the slab. The slab between Joints 15 and 16 is over a balance point between cut and fill and, therefore, cracking has resulted unquestionably from differential volume changes between the fill and cut section and has been augmented by poor subgrade soil characteristics. The cracks in the slabs between Joints 16 and 17, and 19 and 20 were also caused by poor subbase and subgrade conditions.

Slab Movement

The maximum and minimum joint width movements for the winter and summer seasons from 1941 to 1950 are also shown graphically in Figure 40. The character of the graphs indicates: (1) the joint width movements of the sound slabs are very similar in character both with respect to amounts and trends, (2) the sound slabs seem to have a residual contraction which is evidently caused by the relatively high pouring temperature of approximately 80 deg. F; (3) the movement of the joints at either end of the slabs containing cracks or intermediate joints responds in the same manner as normal slabs with intermediate contraction joints in that the amplitude diminishes with time and a progressive residual displacement takes place at the slab ends adjacent to the expansion joint.

VERTICAL MOVEMENT OF PAVEMENT

During the 10-year period covered by this report, three sets of precise elevation measurements have been made over the entire length of the design experimental project. Level measurements representing pavement behavior under winter conditions were taken in 1941 and 1942. Summer level measurements were made in August 1941 and July 1944. All elevations are compared to the base readings which were established soon after construction of the project in 1940.

Vertical Displacement of Pavement

In general, the data in Figure 41 show that the average vertical displacement of the pavement throughout the Test Road has not exceeded 1 inch and in most cases it is less than ½ inch. However, in localized areas changes in elevation of as much as 1. 95 inches occurred during the winter season, and were evidently caused by heaving. Extreme displacements are not shown in Figure 41. Some permanent settlement has occurred ranging on an average less than ½ inch. In some cases the pavement has raised permanently between 0. 2 and 0. 4 inch.

Relative Displacement of Slab Ends

Data showing the average relative displacement of slab ends, or faulting at joints, are presented in Figure 42. The maximum average faulting for the entire test road is less than ⅛ inch. A majority of the sections show faulting of less than 1/16 inch or, for all practical purposes, zero.

Permanent Curling of Slabs

Data on vertical displacement of slab ends with respect to slab center, or in other words slab warping, are presented in Figure 43. The maximum variation of relative vertical displacement is shown as well as the average. The data indicate, in general, that many of the pavement slabs have attained a slight permanent upward warping while others have warped permanently downward.

No attempt has been made at this writing to correlate the displacement phenomena

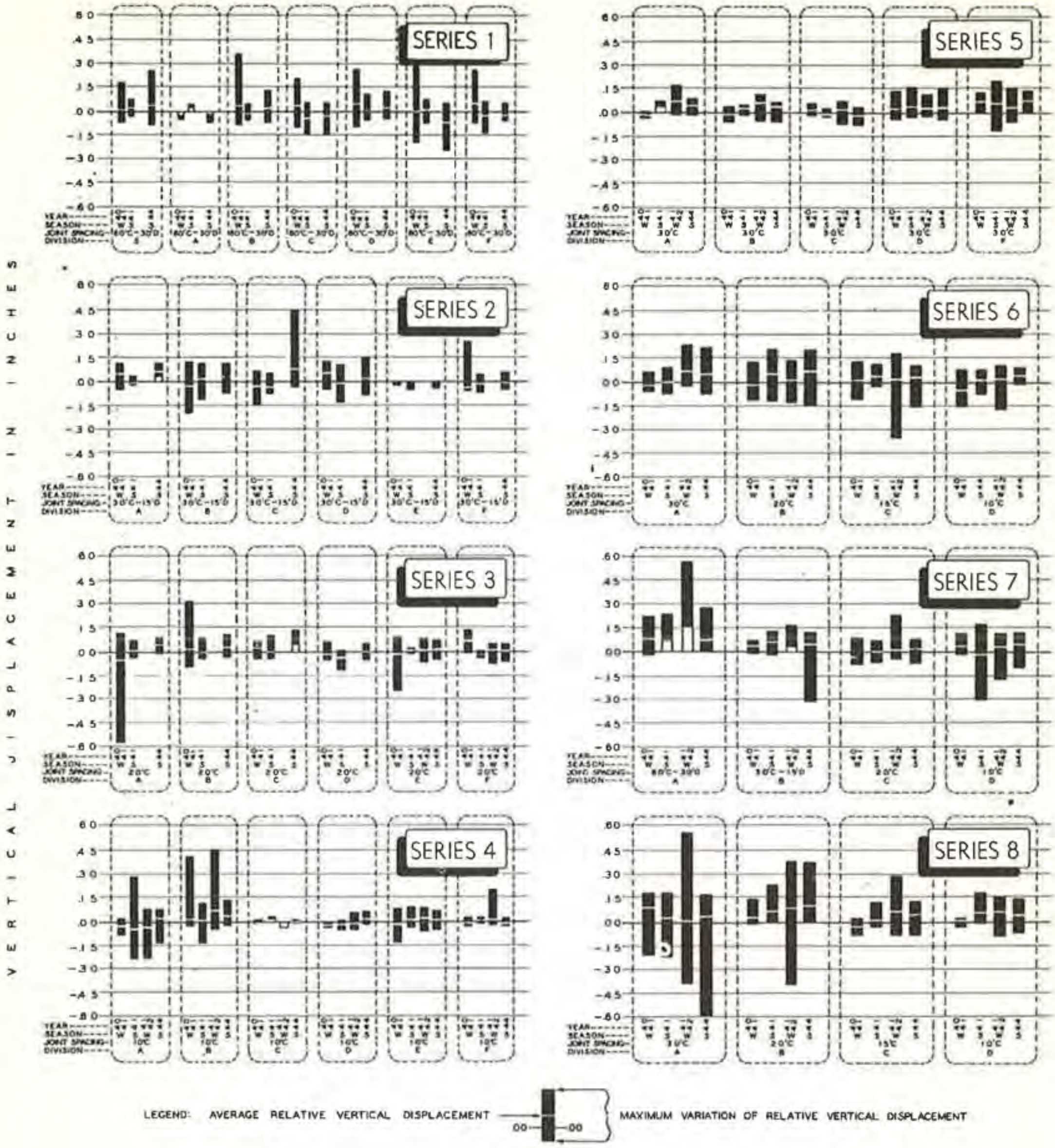


Figure 43. Relative vertical displacement of slab ends with respect to center.

with design features. It is believed that the pavement is far too young at the present time to produce reliable information on performance in relation to certain design features incorporated in the Michigan Test Road.

General Summary

The investigational work associated with the Design Project of the Michigan Test Road has produced to date several results of outstanding importance in the design and construction of concrete pavements. All of these findings which are recapitulated below have been utilized in framing the department's current specifications for concrete pavement construction.

1. The satisfactory performance of long sections of pavement under full restraint indicates that expansion joints are unnecessary except at such places as intersections,

rail crossings, and structures, where excessive compressive stresses introduced by expansion forces are undesirable.

2. Adequate load transfer devices are essential in all joints to preserve the mutual elevation of the abutting slabs.

3. The method of forming a contraction joint by grooving the surface of the pavement and subsequently filling the groove with a good sealing compound is far superior to the method of inserting a premolded bituminous fiber strip.

4. The continuous plate type dowel included in the test road is not satisfactory as a load transfer device.

5. The commercially available asphalt-rubber joint-sealing compounds have greater durability than the mixed-on-the-job asphalt-latex mixtures and are far superior to the straight asphalt or tar products in common use for sealing joints and cracks.

6. Nothing has been learned which would definitely indicate that short slab construction is superior to long slab construction and many advantages are to be gained by the latter practice by way of better riding qualities, lower maintenance costs, and better construction conditions.

7. Results so far indicate that the uniform cross section is equal in performance to that of the thickened edge section, with many obvious advantages.