R-302

The MICHIGAN TEST ROAD

FINAL REPORT on the DESIGN PROJECT

JANUARY 1959

MICHIGAN
STATE HIGHWAY DEPARTMENT
JOHN C. MACKIE
COMMISSIONER

IN COOPERATION WITH
BUREAU OF PUBLIC ROADS
U.S. DEPARTMENT OF COMMERCE

FINAL REPORT - DESIGN PROJECT
MICHIGAN TEST ROAD

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Prepared for Presentation
37th Annual Meeting, Highway Research Board

Report No. 302
Research Project 39 F-7 (2)

Michigan State Highway Department
John C. Mackie, Commissioner
Lansing, January 1959

ACKNOWLEDGEMENTS

The work described in this report has been conducted as part of the Michigan State Highway Department research program vested in the Research Laboratory Division of the Office of Testing and Research, of which W. W. McLaughlin, Testing and Research Engineer, is the head.

The authors gratefully acknowledge the able assistance of many members of the Laboratory staff; in particular, Arthur A. Smith, who was responsible for collecting the bulk of raw data during the life of the road project, and Onto L. Lindy and his staff for processing and preliminary evaluation. Harry D. Cashell, Assistant Highway Engineer, Bureau of Public Roads, observed the project during its construction; his notes and comments have been extremely helpful in preparing this report as well as previous ones on the Design Project.

SYNOPSIS

This report completes the research phase of the Design Project-Michigan Test Road, constructed in 1940 by the Michigan State Highway Department in cooperation with the Public Roads Administration (now Bureau of Public Roads) for the purpose of establishing certain principles in concrete pavement design--in particular, those principles involved in joint spacing and construction methods.

The report contains certain miscellaneous project information pertaining to soil conditions, physical properties of concrete, climate, and traffic, which contribute to an understanding of the findings.

A total of approximately 45,000 joint width measurements were made on more than 850 joints during the 17-year study. Special attention has been given to the statistical analysis of joint width movement in relation to pavement temperature, for each test section. With this information, it was possible to compare various sections and determine the effects on contraction joint width of (a) expansion joint spacing, (b) intermediate warping joints, and (c) contraction joint spacing. The frequency distribution of individual joint width measurements is shown for short slabs with 2,700-ft expansion joint spacing.

The physical performance of the pavement sections is discussed in terms of cracking, spalling, roughness, and joint faulting, in order to relate these to design factors such as joint spacing, pavement thickness and cross section, amount of reinforcement, and load transfer features at joints. Final observations are also given on one of the incidental studies --stress cured concrete pavement.

Conclusions are presented in line with the Bureau of Public Roads' major objectives in this cooperative research study.

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In May 1940, the Michigan State Highway Department authorized construction of an investigational concrete pavement project under regular contract and construction procedure, using the Department's 1940 plans and specifications with necessary supplementals. The specific purposes of this experimental project were twofold: first, to evaluate and establish certain fundamental design principles of concrete pavement construction, and second, to determine under field conditions the effects of certain factors on the durability of concrete, particularly in relation to scaling.

The Michigan Test Road was divided into two experimental sections. One, designated the Design Project, was 10.1 mi in length and coincides generally with the Bureau of Public Roads Plan and Procedure for the construction of experimental roads as submitted to various state highway organizations in 1940, but was more comprehensive in scope. The Design Project was one of a group of six such test roads built throughout the United States, the others being in California, Kentucky, Minnesota, Missouri, and Oregon. The other experimental section, called the Durability Project, was 7.1 mi in length and was included by the Department in the construction of the Test Road to supplement laboratory studies on concrete durability, especially in regard to scaling.

The purpose and scope of the entire research program were reported in a bulletin titled "The Michigan Test Road," published by the Department in July 1942; subsequent to the release of this publication on both the Design and Durability Projects, four reports devoted exclusively to the Design Project were issued which should be noted here. The first of these may be found in <u>Proceedings</u>, Highway Research Board, Vol. 20 (1940). A preliminary progress report describing only the Design Project is included in Highway Research Board Report No. 3-B (1945). A nine-year progress report was published by the Department in August 1950, and a ten-year report appears in Highway Research Board Report No. 17-B (1956). A final report on the Durability Project is being prepared.

Because of these earlier publications, repetition of certain basic information purposely has been avoided in this report, except where necessary for better understanding of the results. The reader is cautioned that tables and figures presented here include some revision and minor correction of similar material in these earlier publications. These revisions, however, do not substantially alter the basic principles and conclusions.

Due to advanced scaling of the pavement surface, resulting in rough riding conditions, certain local areas of the Test Road were resurfaced prior to the eventual complete resurfacing of the Design Project with bituminous concrete in 1957. These local areas were: Sta 222+70 to 225+10 (part of Section 10B-2) resurfaced in 1948; Sta -5+27 to 27+10 (part of Sections 4D and 4F and all of 4E) resurfaced in 1953, and Sta 38+91 to 225+10 (part of Section 4D and all of Series 5 through 10) resurfaced in 1956. Since the Design Project is now completely resurfaced, this report will be the last on that project and will summarize observational data from 17 years of service.

It may be mentioned at this point that according to the Bureau of Public Roads Division of Physical Research, the three most important objectives in establishing the six experimental roads were:

"1. To determine whether expansion joints could be eliminated or spaced at much greater intervals in plain concrete pavements with closely spaced contraction joints, than was accepted practice at the time that this investigation was started, without causing blowups or other detrimental effects to the pavement.

"2. To determine whether aggregate interlock could be depended upon to prevent faulting in plain concrete pavement with closely spaced weakened plane contraction joints and expansion joints eliminated or spaced at long intervals.

"3. To compare the performance of reinforced concrete pavement and plain concrete pavement of conventional designs with different expansion joint arrangements."

Therefore, in analyzing the data for presentation, the objectives above have been kept in mind, together with pertinent factors of particular interest to the Department, such as joint design, pavement cross section, steel reinforcement, uniform thickness versus balanced cross section, and pavement performance as related to construction factors. This information will be presented under the headings "Miscellaneous Project Information," "Joint Spacing," "Pavement Design," and "Incidental Studies."

MISCELLANEOUS PROJECT INFORMATION

The test areas designated as Series 1 to 12 are described in Table 1. To facilitate study of particular design features, each series has been further divided into sections and subsections designated by letters and numerals, respectively. During and after construction of the pavement surface, certain important physical data were procured which might be directly or indirectly associated with general behavior of the pavement slabs. Such information included general soil conditions and subbase construction operations, climatic data, physical properties of the concrete, and traffic conditions.

General Soil Conditions

The subgrade materials were primarily well-drained sandy or gravelly soils with the exception of two areas, from Sta 88+00 to 129+99 and from Sta 170+00 to 225+06, where it was necessary to construct a 12-in. sand subbase over existing subgrade material. Although, in general, granular subbase and subgrade materials fell into Bureau of Public Roads soil classification A-3, subgrade soil material between the stations cited met Bureau classifications for A-4 and A-6 soils. The physical properties of granular subgrade and subbase soil materials from four representative locations are given in Table 2. When concrete was placed, soil density at a point 9 in. below the bottom of the slab ranged from 103 to 113 lb per cu ft. Moisture content of the soil at that time varied from 4.2 to 7.6 percent of the soil's dry weight.

Subgrade performance has been satisfactory throughout the project with the exception of several frost heave areas which developed in Series 6 and 9. The effect of frost heave on slab performance was discussed in the 10-year report on the Design Project.

Physical Properties of the Concrete

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

TABLE 1
SUMMARY OF TEST AREAS - DESIGN PROJECT

Test Area Designation						Joint Spacing in Feet			Load Transfer Type			Filler and Seal	
Series	Section	Subsection	Length of Section in ft	Pav't Thickness inches	Reinf lb/100 sq ft	Expansion	Contraction	Warping	Expansion (1)	Contraction (2)	Warping (3)	Expansion Joint (4)	Special Factors Under Study
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 - 3/4" x 15" Dowel Bar Expansion Joint Assembly. Dowels at 15" spacing.
Type TE - Thickened Edge 1 1/4" x 18" Corner Dowel Bar Expansion Joint Assembly. Dowels 9" from slab edge. pavement edge.
Type CB - 1 - Unthickened Edge. 1" 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 - 3/4" x 15" Dowels at 15" spacing, premolded filler.
Type 1B - 3/4" x 15" Dowels at 15" spacing, groove and poured seal.
Type 2A - 3/4" x 15" Dowels at 15" spacing, premolded filler, metal parting strip at bottom.
Type 2B - 3/4" x 15" Dowels at 15" spacing, groove and poured seal, metal parting strip at bottom.
Type 3 - 3/4" 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 - 1" x 18" Dowels at corners, 9" from slab edge, premolded filler.
Type 6 - Aggregate Interlock. No Dowels.

(3) WARPING JOINT CONSTRUCTION:
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 SUBBASE AND SUBGRADE GRANULAR SOIL MATERIAL

Properties	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	10.3
Field moisture equivalent, %	19	18	20	17
Moisture, bottom inch of rise, %	24.9	23.9	23.0	20.2
Moisture, top inch 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	
	12-in. cylinders 28 days	6-in. dia. cores 21 months	6- by 8- by 24-in. beams		10^6 psi	
			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-inch average) . . . 2.03 inches

Weight per Cubic Foot . . . . 153 pounds

Climatological Information

Figure 1 shows average daily temperature variation from 1941 to 1957, and Figure 2 presents the average daily temperature for the same period. Temperatures in this report are expressed in degrees Fahrenheit. Daily temperature fluctuations in the winter ranged from a minimum

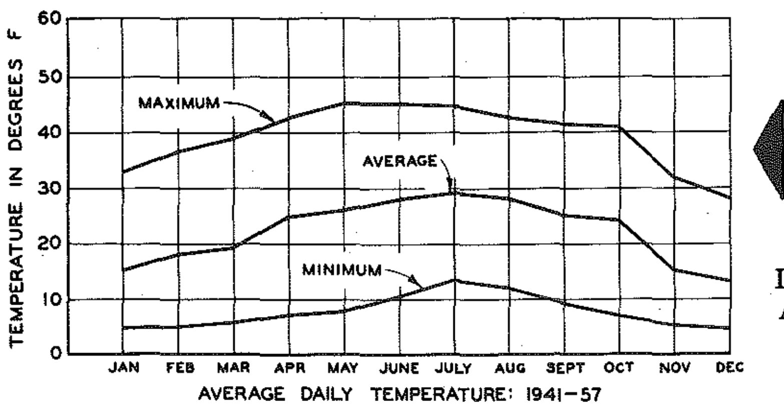


Figure 1.
Daily Temperature,
Average Variation:
1941-57

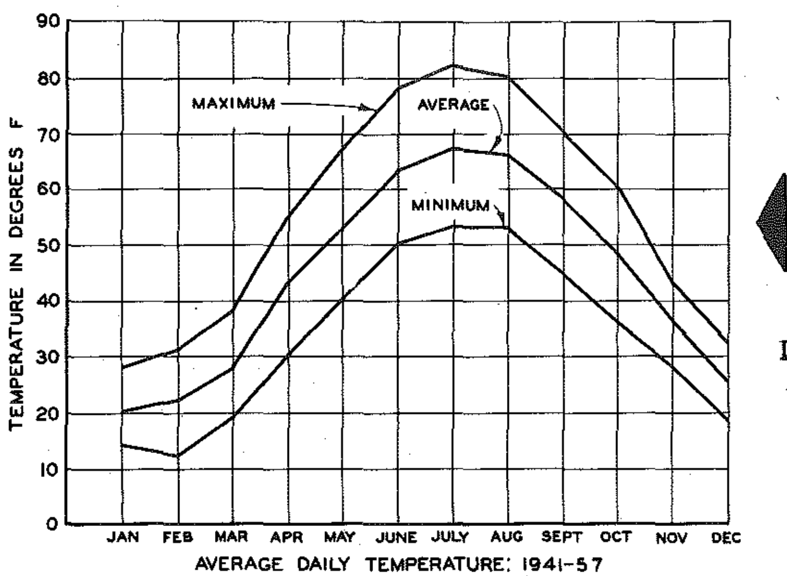


Figure 2.
Daily Temperature
Average: 1941-57

of 4 deg to a maximum of 39 deg, or an average of about 17 deg; during the summer, the range was from a low variation of 9 deg to a high of 45 deg, the average being about 27 deg. Average daily temperature (Figure 2) varied from 20 deg in winter to close to 67 deg in summer, a total average annual change of 47 deg.

Total annual precipitation, 1941-57 inclusive, is given in Figure 3. The data indicate an average annual rainfall of 31.92 in. It may be noted that yearly variation from the 17-year average is slight, indicating fairly uniform moisture conditions through the life of the project.

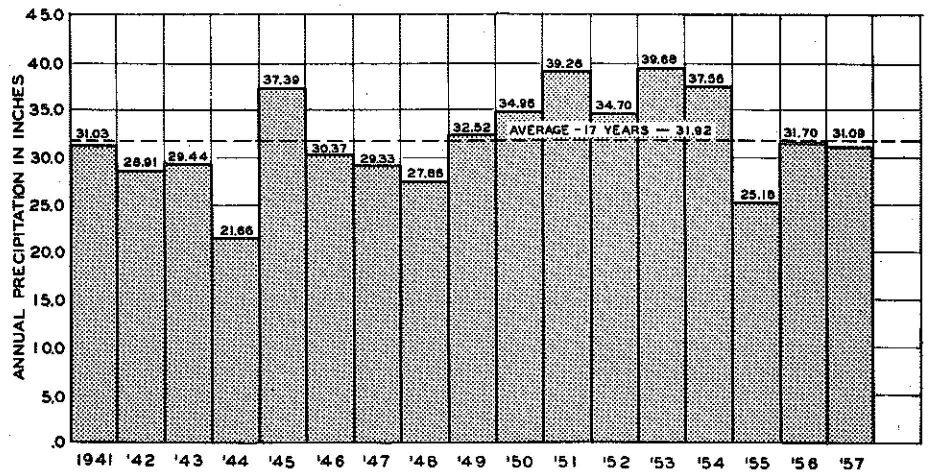


Figure 3. Annual Precipitation

Traffic Characteristics

Automatic recording equipment was installed at the Test Road to obtain a continuous daily record of traffic flow. Traffic classification surveys were made quarterly--in January, April, July, and October--covering a 6-hr period daily for five days. The 6-hr periods were rotated around the clock in order that data representing the 24-hr day for

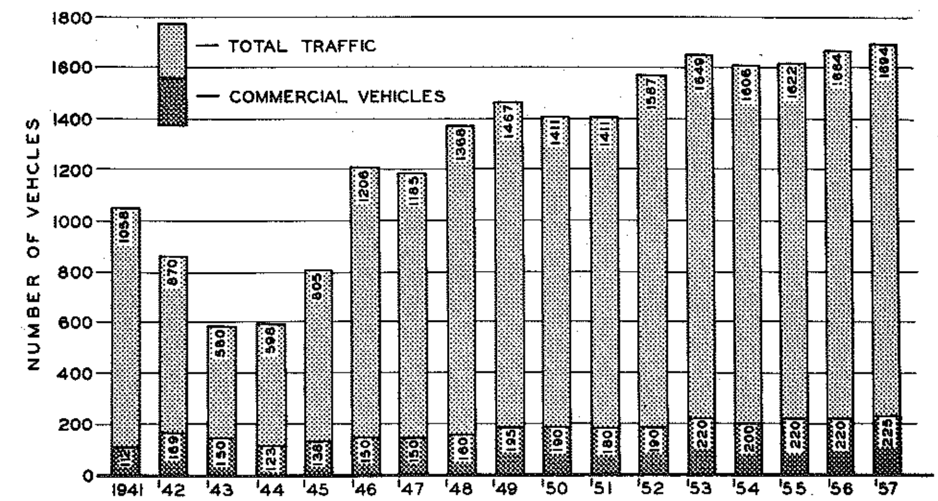


Figure 4. Average Daily Traffic

the different seasons could be obtained for each year. For one year, 1950-1951, this classification procedure was changed to a continuous 24-hr period each month. Similar surveys elsewhere indicate that such a procedure gives better results. During the surveys, the frequency of commercial vehicles was recorded, with axle loads and spacings. Wheel loads were obtained by means of portable loadometers.

Annual average daily traffic flow from 1941 to 1957 is shown in Figure 4. With exception of the war years 1942-45, total traffic increased slightly. Commercial traffic generally increased at a rather uniform rate throughout the 17 years and by the end of this period had about doubled. The average monthly totals for passenger and commercial traffic are shown in Figure 5, which clearly demonstrates the seasonal pattern of total traffic flow over the project.

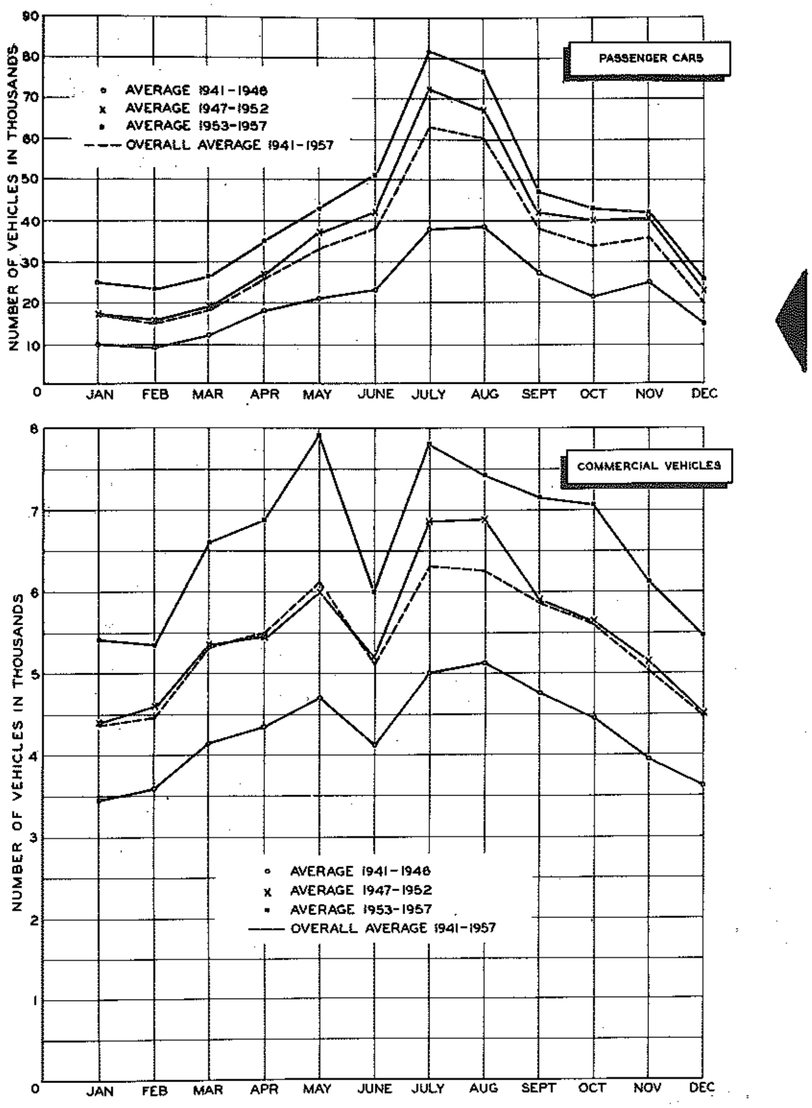


Figure 5.

Average Monthly Traffic

The accumulated tonnage of traffic estimated to have passed over the road during 17 years of service is shown in Figure 6. If similar information were available from the other five state experimental projects, it might be useful in comparing relative traffic loads.

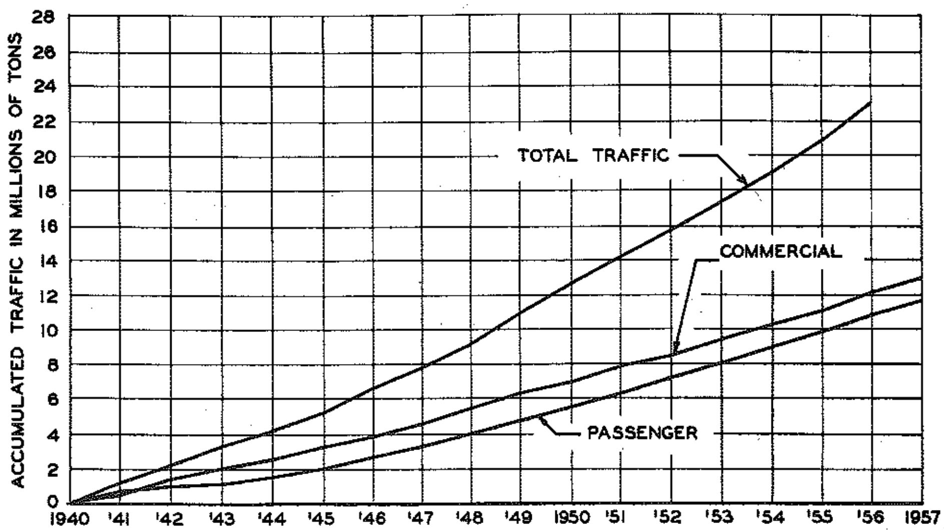


Figure 6. Accumulated Tons of Traffic

The axle load frequency on the Michigan Test Road, averaged for the 17 years, is presented graphically in Figure 7. For comparison, a similar axle load frequency curve is shown for 1955 commercial traffic on

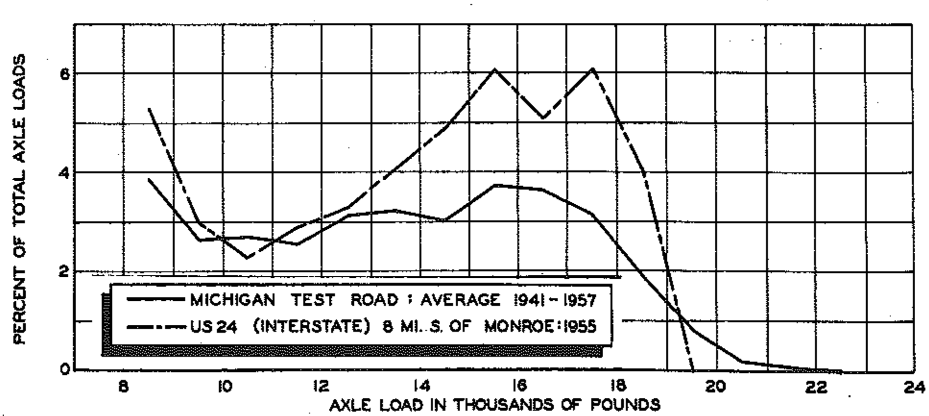


Figure 7. Axle Load Frequency

heavily travelled Interstate route US 24, 8 mi south of Monroe, Mich. A further comparison is made in Figure 8, showing the percentage of total commercial axle loads in excess of any given weight--one quarter of the axle loads exceed 10,800 lb on the Michigan Test Road, while on US 24, one quarter exceed 14,200 lb.

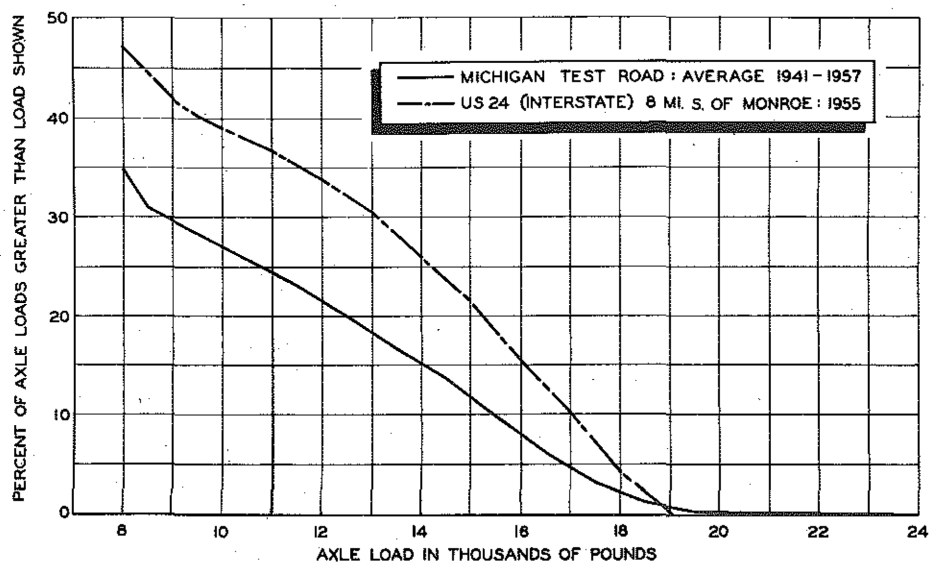


Figure 8. Percent of Axle Loads Exceeding Weight Shown

Numerical data concerning classification of annual average daily traffic are given in Table 4, and Table 5 contains numerical values for annual average daily wheel load distribution.

Year		Classification		
		Passenger	Commercial	Total Traffic
1941	No.	946	112	1058
	%	89.4	10.6	100.0
1942	No.	701	169	870
	%	80.6	19.4	100.0
1943	No.	430	150	580
	%	74.1	25.9	100.0
1944	No.	475	123	598
	%	79.4	20.6	100.0
1945	No.	667	138	805
	%	82.9	17.1	100.0
1946	No.	1056	150	1206
	%	87.6	12.4	100.0
1947	No.	1035	150	1185
	%	87.3	12.7	100.0
1948	No.	1208	160	1368
	%	88.3	11.7	100.0
1949	No.	1272	195	1467
	%	86.7	13.3	100.0
1950	No.	1221	190	1411
	%	86.5	13.5	100.0
1951	No.	1231	180	1411
	%	87.2	12.8	100.0
1952	No.	1397	190	1587
	%	88.0	12.0	100.0
1953	No.	1429	220	1649
	%	86.7	13.3	100.0
1954	No.	1406	200	1606
	%	87.5	12.5	100.0
1955	No.	1402	220	1622
	%	86.4	13.6	100.0
1956	No.	1444	220	1664
	%	86.8	13.2	100.0
1957	No.	1469	225	1694
	%	86.7	13.3	100.0

TABLE 4. CLASSIFICATION OF ANNUAL AVERAGE DAILY TRAFFIC

Wheel Load	1945		1946		1947		1948		1949		1950	
	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Under 4000	595	60.65	678	65.76	888	59.96	746	60.55	805	61.04	3168	63.81
4000 - 4499	32	3.26	31	3.01	55	3.71	57	4.63	44	3.34	173	3.49
4500 - 4999	25	2.55	19	1.84	36	2.43	40	3.25	32	2.43	159	3.20
5000 - 5499	31	3.16	25	2.43	46	3.11	51	4.14	30	2.27	164	3.30
5500 - 5999	24	2.45	37	3.59	43	2.90	31	2.52	41	3.11	165	3.32
6000 - 6499	36	3.67	42	4.07	40	2.70	36	2.92	58	4.40	192	3.87
6500 - 6999	39	3.98	36	3.49	43	2.90	39	3.17	53	4.02	164	3.30
7000 - 7499	39	3.98	30	2.91	49	3.31	47	3.82	45	3.42	186	3.75
7500 - 7999	58	5.91	39	3.78	67	4.52	46	3.73	53	4.02	182	3.67
8000 - 8499	50	5.10	33	3.20	64	4.32	41	3.33	61	4.63	145	2.92
8500 - 8999	26	2.65	26	2.52	62	4.19	36	2.92	51	3.87	122	2.46
9000 - 9499	18	1.83	19	1.84	55	3.71	28	2.27	22	1.67	86	1.73
9500 - 9999	8	0.81	14	1.36	22	1.49	17	1.38	16	1.21	34	0.68
10000 - 10499			2	0.20	6	0.41	9	0.73	4	0.29	14	0.28
10500 - 10999					4	0.27	5	0.40	2	0.14	4	0.08
11000 - 11499					1	0.07	1	0.08	1	0.07	2	0.04
11500 - 11999							2	0.16	1	0.07	0	0.00
12000 - 12499											0	0.00
12500 - 12999											1	0.02
13000 - 13499											3	0.06
13500 - 13999											1	0.02
46500 - 46599												
Total Axles	981	100.00	1031	100.00	1461	100.00	1232	100.00	1319	100.00	4965	100.00
Total Vehicles	348		375		564		414		466		1771	
Ratio: $\frac{\text{Axles}}{\text{Vehicles}}$		2.82		2.75		2.63		2.98		2.83		2.80

Wheel Load	1951		1952		1953		1954		1955		1956		1957	
	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%	No.	%
Under 4000	1066	61.55	162	71.06	195	71.18	189	76.52	182	70.54	248	69.09	197	72.69
4000 - 4499	72	4.16	13	5.70	6	2.19	10	4.05	9	3.49	19	5.29	16	5.90
4500 - 4999	53	3.06	7	3.07	4	1.46	9	3.65	9	3.49	9	2.51	5	1.85
5000 - 5499	60	3.46	7	3.07	2	0.73	4	1.62	1	0.39	10	2.79	7	2.58
5500 - 5999	66	3.81	6	2.63	3	1.09	5	2.02	7	2.71	8	2.23	4	1.48
6000 - 6499	81	4.68	4	1.75	4	1.46	7	2.83	4	1.55	8	2.23	6	2.21
6500 - 6999	72	4.16	5	2.19	8	2.92	6	2.43	7	2.71	4	1.11	7	2.58
7000 - 7499	71	4.10	6	2.63	10	3.65	1	0.41	1	0.39	6	1.67	4	1.48
7500 - 7999	65	3.75	3	1.32	6	2.19	2	0.81	7	2.71	7	1.95	5	1.84
8000 - 8499	61	3.52	4	1.75	15	5.47	4	1.62	4	1.55	14	3.90	2	0.74
8500 - 8999	38	2.19	6	2.63	14	5.11	4	1.62	16	6.20	17	4.73	5	1.85
9000 - 9499	12	0.69	4	1.75	4	1.46	5	2.02	5	1.94	5	1.39	9	3.32
9500 - 9999	6	0.35			3	1.09	0	0.00	5	1.94	3	0.83	1	0.37
10000 - 10499	6	0.35					1	0.40	0	0.00	0	0.00	1	0.37
10500 - 10999	3	0.17							1	0.39	1	0.28	0	0.00
11000 - 11499													1	0.37
11500 - 11999													1	0.37
12000 - 12499														
12500 - 12999														
13000 - 13499														
13500 - 13999														
46500 - 46599			1	0.45										
Total Axles	1732	100.00	228	100.00	274	100.00	247	100.00	258	100.00	359	100.00	271	100.00
Total Vehicles	620		90		101		90		98		115		91	
Ratio: $\frac{\text{Axles}}{\text{Vehicles}}$		2.79		2.53		2.71		2.74		2.63		3.12		2.98

TABLE 5. ANNUAL AVERAGE WHEEL LOAD DISTRIBUTION

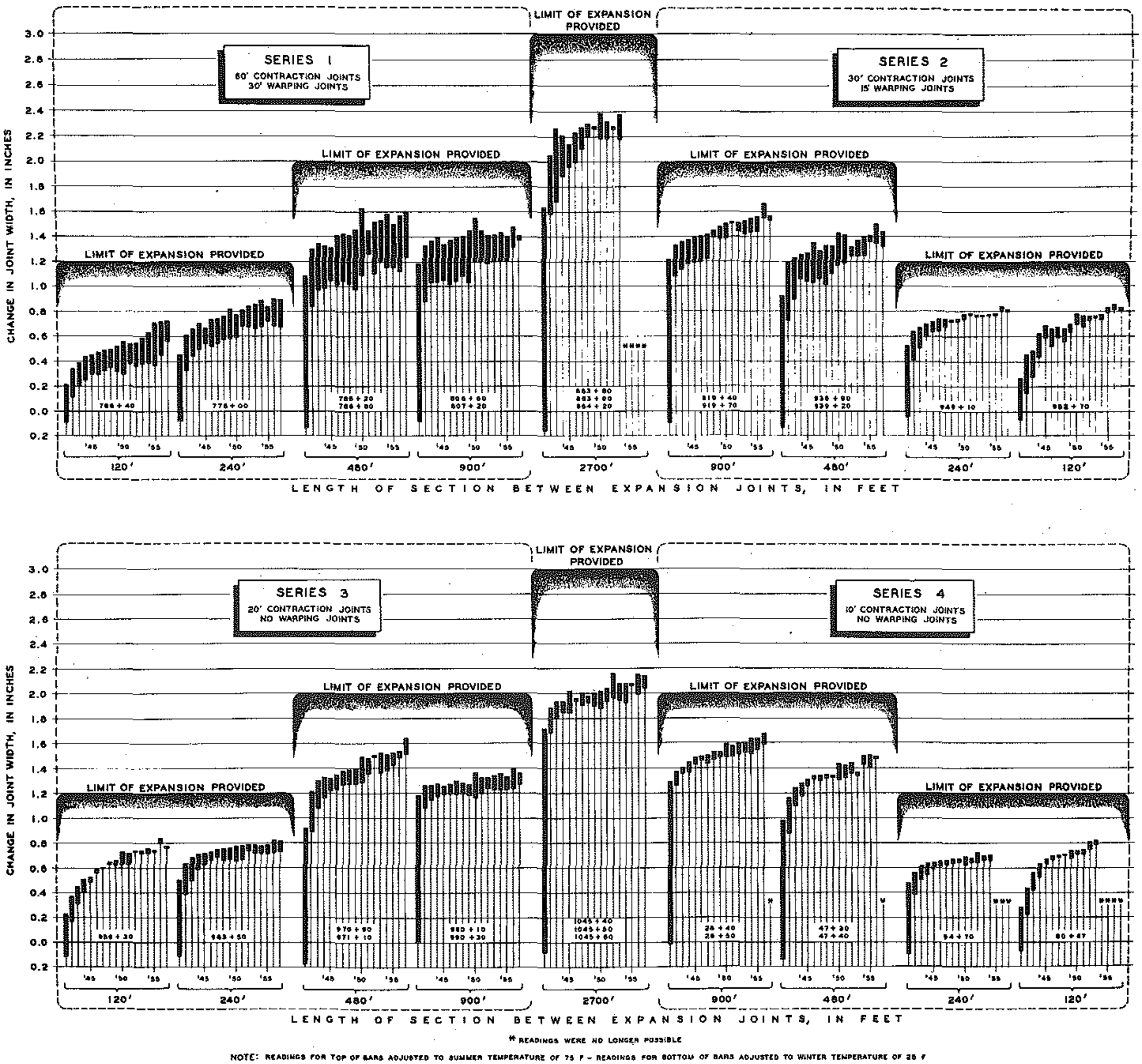


Figure 9. Annual and Progressive Changes in Expansion Joint Widths

JOINT SPACING

Series 1, 2, 3, and 4 were designed primarily to study joint width movement in relation to slab length and expansion joint spacing, in conjunction with the experimental road program of the Bureau of Public Roads. Initial measurements of joint width and slab position were made during the summer and fall of 1940, immediately after completion of each series, these readings being used as references in determining subsequent displacements. Thereafter, seasonal and daily readings were taken as near as possible at the same time of day during all observation periods. But, because two to four weeks were required to make all measurements, it was expected that normal daily climatic fluctuations during that time would, to some extent, influence the seasonal and daily joint width measurements. In addition, slab curling and warping would have certain effects on joint width readings. The effects of these day-to-day changes in slab conditions during the observation periods have not been considered in interpreting data for this report.

The particular days for seasonal joint width measurements depended largely on weather conditions. In general, spring readings were taken in late April or early May, summer measurements in July or August, fall readings in October and November, and winter readings any time from January to March. Winter readings were made when temperatures were moderate, and the pavement surface sufficiently free of snow and ice to permit measurements.

Joint width measurements for slabs 50 to 100 ft in length, from another Michigan experimental project, have been introduced into this report to supplement corresponding data for the Design Project, to lend greater significance to the Test Road data, and to make the findings more applicable to pavement design problems in general.

Transverse joint types included in the joint spacing study are expansion, contraction, and hinge or warping joints.

Expansion Joints

Seasonal change in expansion joint widths for several sections in Series 1, 2, 3, and 4, together with progressive or permanent change, are presented graphically in Figure 9 for the years 1941-57 inclusive. These graphs also show the relationship between joint width changes, the section lengths between expansion joints, and amount of expansion space provided. Unless otherwise stated, only those expansion joints separating sections of equal length were considered in plotting the graphs. Where relief sections are involved--that is, sections consisting of two or more expansion joints, separated by short slabs of concrete--individual expansion joint movements were combined algebraically to form a single value representing a joint of equivalent width.

The joint width readings in Figure 9 have been adjusted to represent an average pavement temperature of 75 deg in summer and 25 deg in winter, using corrections derived from daily movements.

Several significant facts are revealed by the graphs in Figure 9:

1. In most cases, sections contracted sufficiently during the first winter to cause expansion joint openings slightly in excess of the expansion widths originally provided.

2. Without exception, all sections moved most during the first year.

3. The annual amplitude of joint width movement diminished with time.

4. All expansion joints show progressive permanent reduction in width, resulting in gradual closing to the extent that after 17 years the sections absorbed about 60 to 80 percent of the expansion space provided.

5. As might be expected, the longer sections produced the greatest changes in joint width during the first year, although amplitude of annual joint movement after the first year was comparable to that of the shorter sections.

6. The amplitude of yearly movement was least for the sections with 10-ft and greatest for those with 60-ft contraction joints. This phenomenon indicates that a considerable amount of section movement was absorbed by the more numerous contraction joints present in sections with 10-ft spacing.

Contraction Joints

The data obtained from summer and winter measurements for joints in each section were plotted with joint opening as the dependent variable and concrete temperature as the independent variable. In test sections with expansion joint spacings up to and including 480 ft, all contraction joints in each section were considered collectively. However, in test sections longer than this, data for joints near section ends were kept separate from data for joints near the middle of sections.

Plotting the data indicated that a linear relationship existed between contraction joint opening and concrete temperature, and the line of regression relating these two variables was obtained by the statistical method of least squares. During the first three years, joints opened less and the openings were less consistent than during the following 14 years; thus, statistical analysis was based on joint measurements from the fourth through the seventeenth years. In general, a section's averaged joint readings at a given temperature scatter only slightly around the line of regression. But when all the individual joint readings for a given section are plotted with respect to concrete temperature, the scatter is much greater, as would be expected.

Effects of Time and Temperature: To show both the range in joint opening and the effect of time, the individual joint readings have been plotted from four sets of data. Figures 10 and 12 show data for contraction joints at the middle of Test Road Sections 4F and 3F (10- and 20-ft contraction joint spacings respectively, no intermediate warping joints, and 2,700-ft expansion joint spacing), while Figures 11 and 13 show the data from the Grand Ledge-Mulliken Test Road (50- and 100-ft contraction joint spacing and no intermediate warping joints or expansion joints). These graphs indicate the scatter of individual joint opening measurements at given concrete temperatures, and the effect of time over 17- and 12-year periods respectively, for these two test roads. Lines are also shown at one standard deviation of the errors of estimate on either side of the line of regression. The chances are 68 in 100 that an individual joint opening at a given temperature will be between the limits established by these lines.

In Figures 10, 11, 12, and 13, the correlation coefficient for temperature and joint width opening ranges from -0.602 to -0.889, where "0" signifies no correlation and "-1" signifies perfect correlation between increasing joint width opening and decreasing temperature. It should be noted that winter average joint openings (25 deg) were approximately 0.05, 0.08, 0.20, and 0.41 in. for contraction joint spacings of

10, 20', 50', & 100' slabs

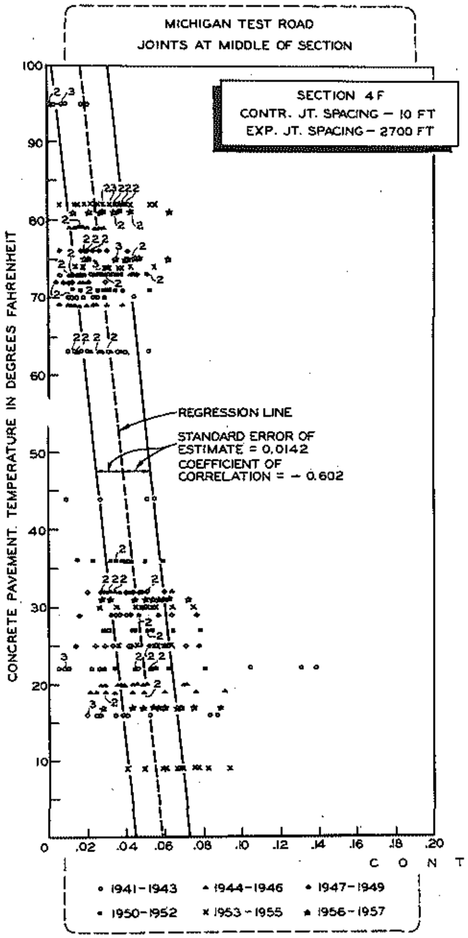


Figure 10.

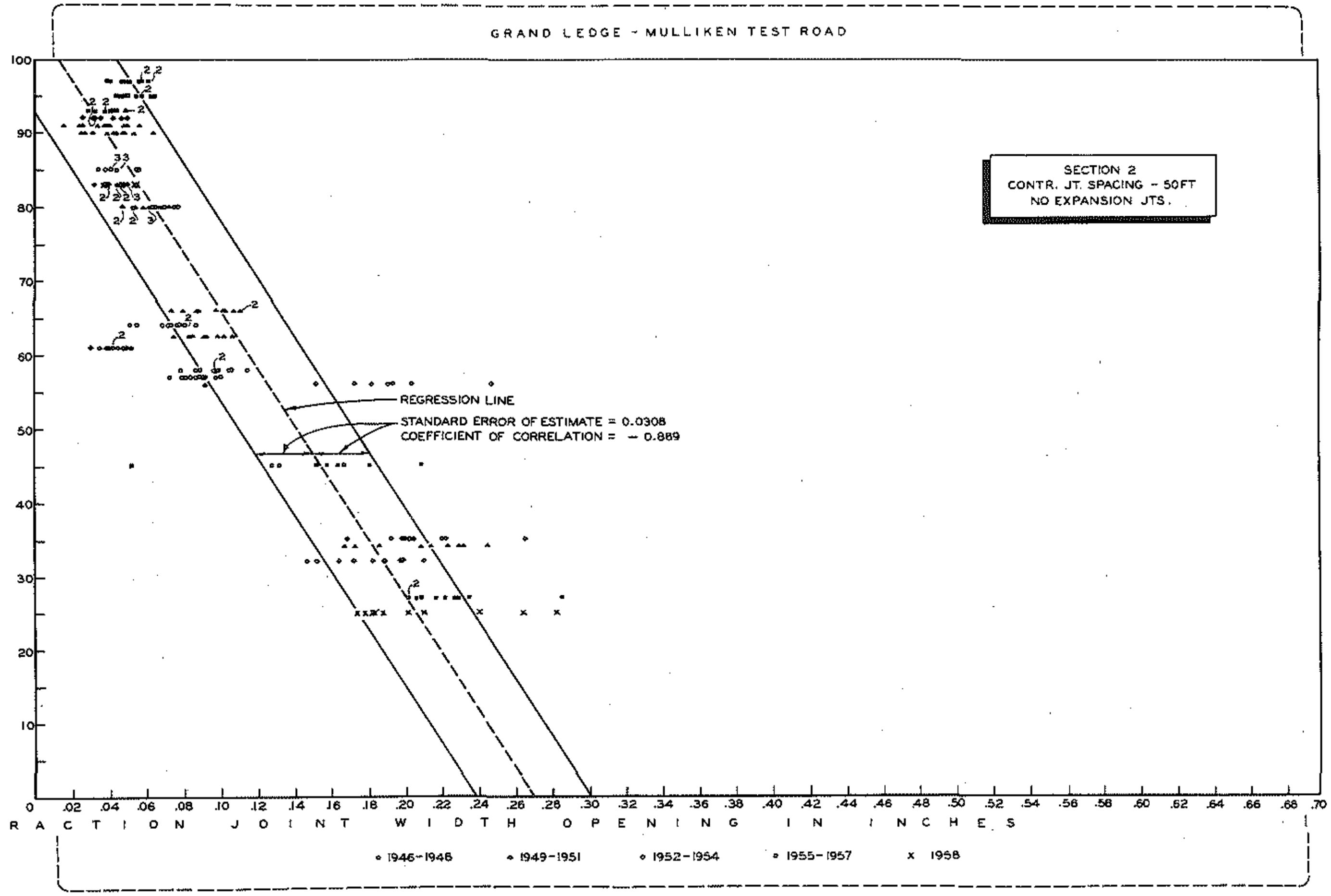


Figure 11.

Joint Opening Versus Temperature

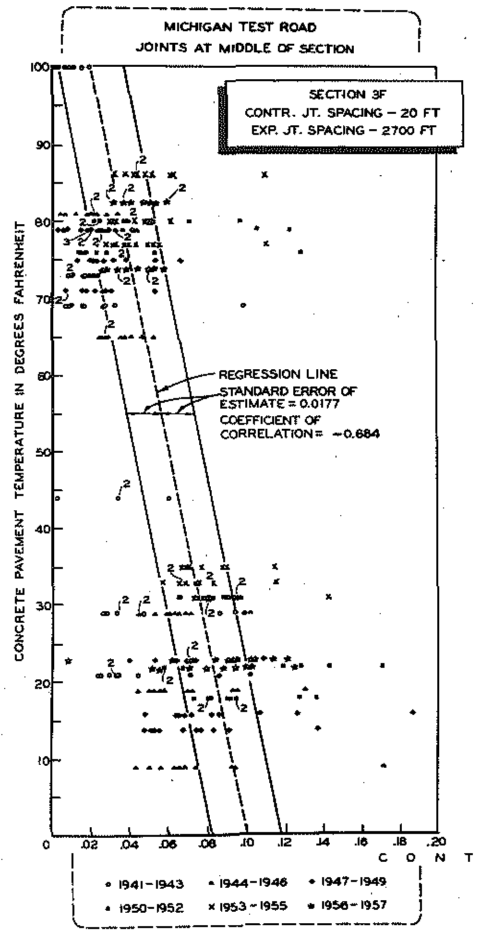


Figure 12.

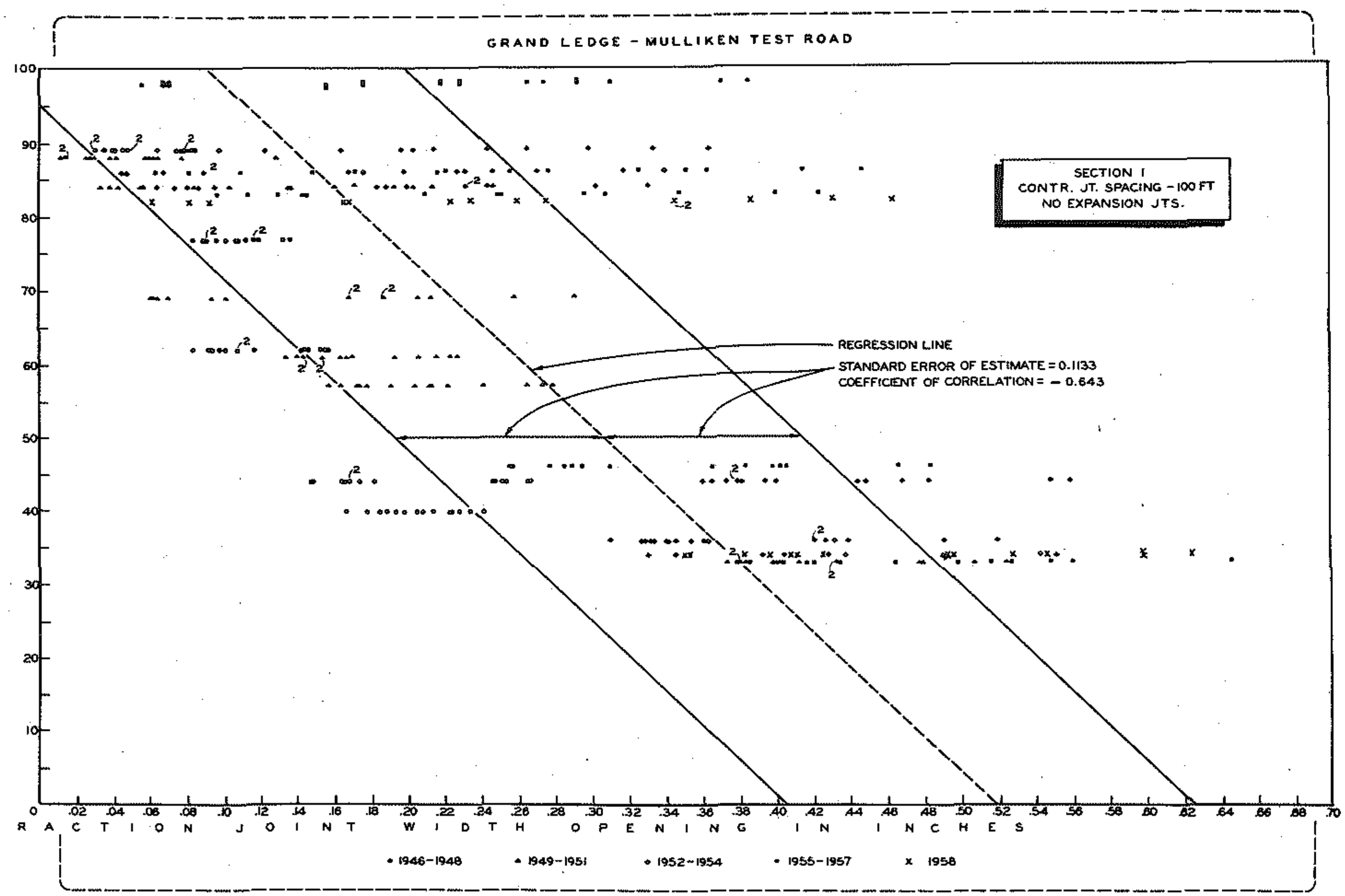
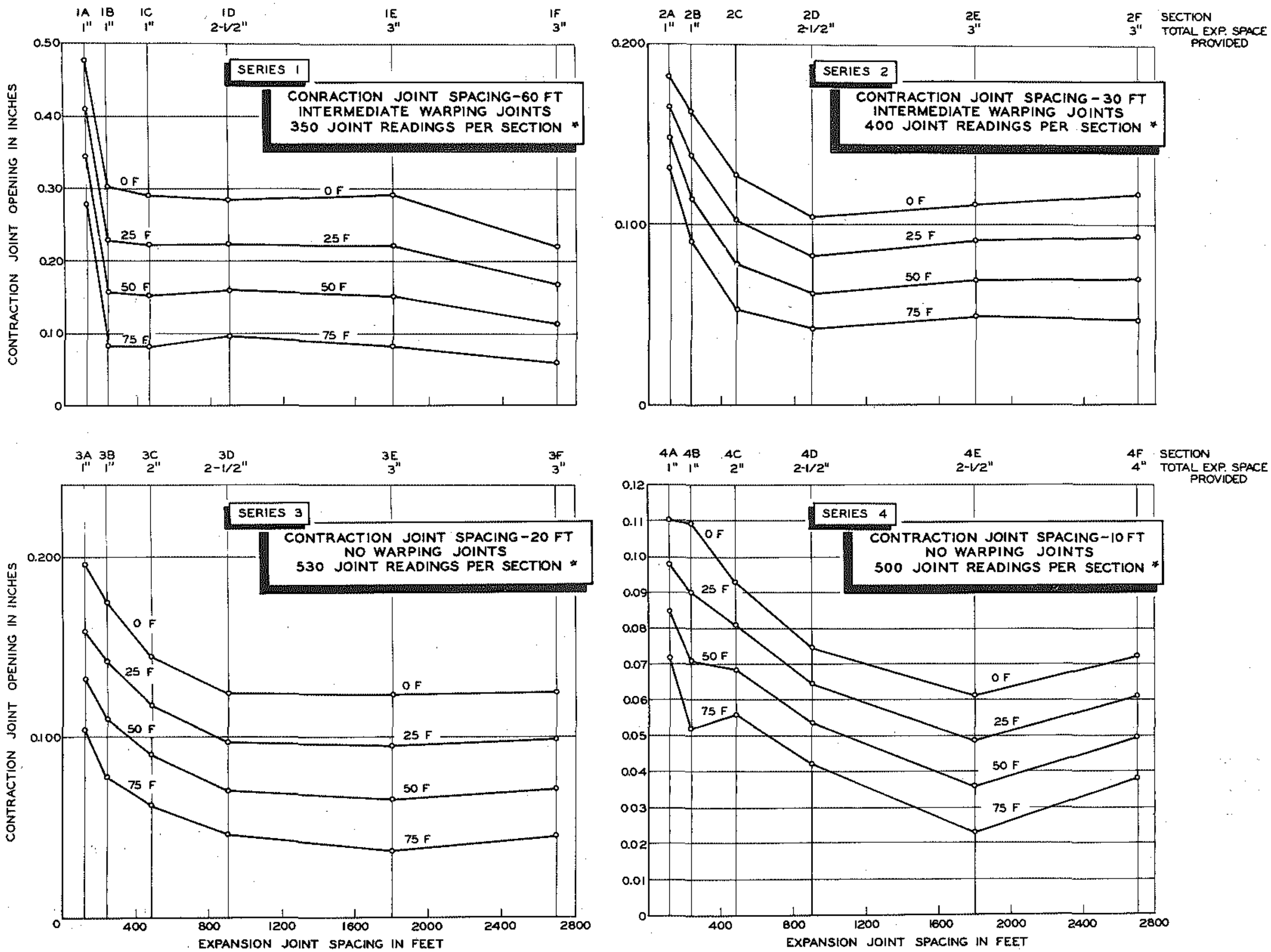


Figure 13.

Joint Opening Versus Temperature



*— DATA OBTAINED FROM STATISTICAL ANALYSIS
OF LINE OF REGRESSION FOR EACH SECTION

Figure 14. Effect of Expansion Joint Spacing on Contraction Joint Opening

10, 20, 50, and 100 ft respectively. In general, joint opening appeared to increase slightly with age for each of the four slab lengths under discussion. This is illustrated by the fact that, generally, readings for the first few years are on the low side of the line of regression and for the last few years, on the high side.

Effect of Expansion Joints. By comparing data from Sections A through F of Series 1, 2, 3, and 4, it is possible to determine the effect of expansion joint spacing on contraction joint width openings for several slab lengths. In Figure 14, the data for various curves were obtained by averaging readings from all instrumented joints in each test section. There was some difference in the joint width movements near the end of a section (close to an expansion joint or relief section), and for the joints near the middle of a section. Joints near section ends had openings which averaged 2, 12, and 28 percent higher than joints in the middle of the section, for the 900-, 1,800-, and 2,700-ft expansion joint spacings, respectively. Points shown on these graphs for various temperatures were obtained from lines of regression based on statistical analysis of joint opening versus temperature. In every case, contraction joint opening decreased markedly as expansion joint spacing increased from 120 to 240 ft, and with one inch of total expansion space. Generally, the decrease in contraction joint opening continues to an expansion joint spacing of 900 ft (total expansion space of 2 1/2 in.), but then stays rather uniform for 900-, 1,800-, and 2,700-ft expansion joint spacings (total expansion space of 2 1/2, 3, and 3 in. respectively). An exception to this rule is Series 4, where the pattern is somewhat different, because total expansion space provided for 1,800- and 2,700-ft expansion joint spacings was 2 1/2 and 4 in. rather than 3 in. as in Series 1, 2, and 3.

Effect of Warping Joints. Several sections of Series 1 and 2 were instrumented at intermediate warping joints to obtain joint width measurements. At all these instrumented joints, the openings increased progressively with time. From readings in Series 1 and 2, it was possible to determine the effect of warping joint widths upon contraction joint openings. In Figure 15, actual contraction joint openings are plotted for four temperatures from the lines of regression for Sections B and F of Series 1 through 4. In order to determine the adjusted contraction joint width for Series 1 and 2, with intermediate warping joints, the intermediate and contraction joint widths were added, and an adjusted line of regression obtained approximating the contraction joint opening which would have occurred without the intermediate warping joints. This is shown by the dotted lines in Figure 15.

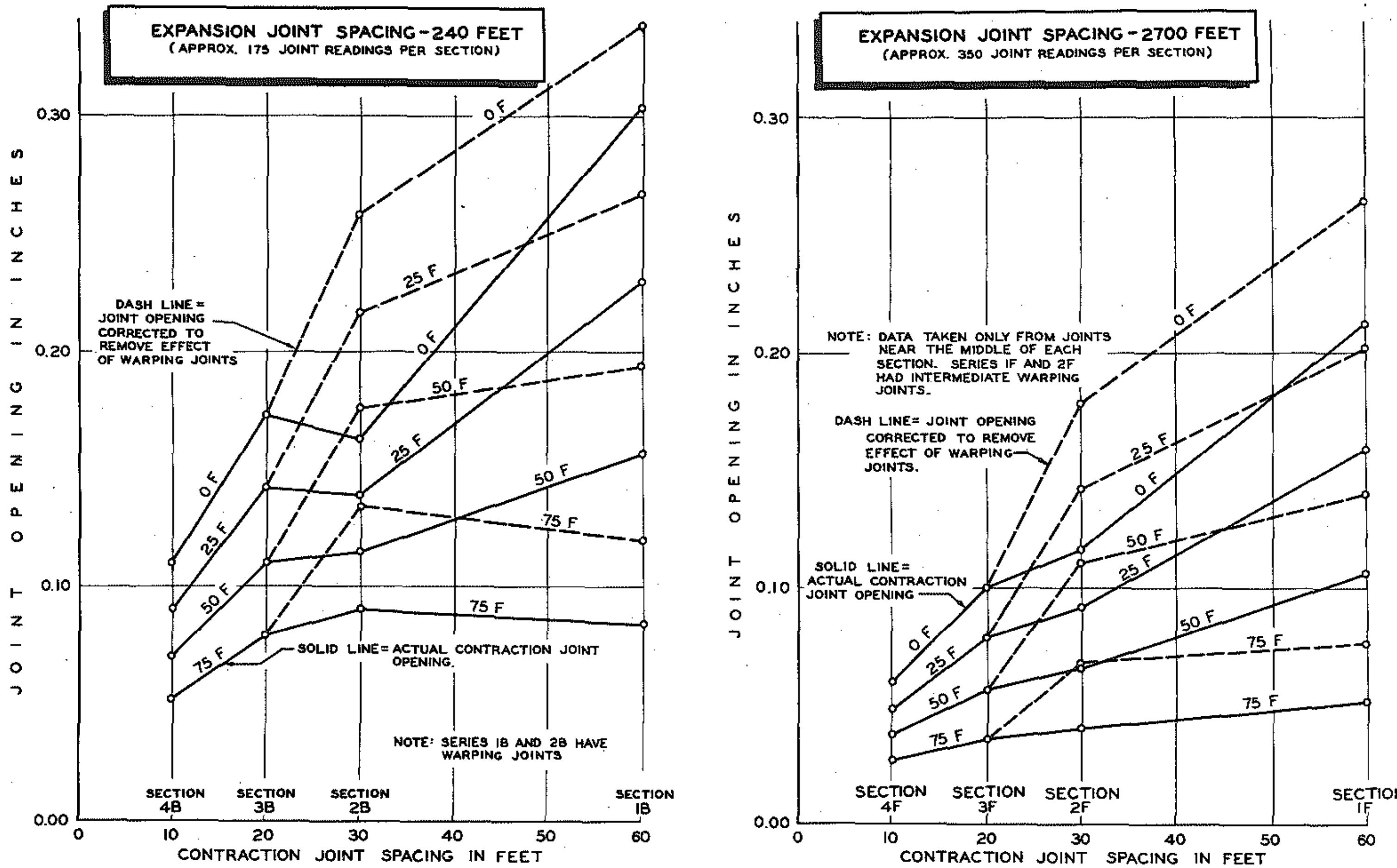


Figure 15. Effect of Contraction Joint Spacing on Contraction Joint Opening

Effect of Slab Length. To determine average contraction joint openings over a considerable range of contraction joint spacing, without the complicating effects of expansion or intermediate warping joints, the Michigan Test Road data from Sections 3F and 4F were again supplemented by data from 50- and 100-ft contraction joint spacings on the Grand Ledge-Mulliken Test Road. In Sections 3F and 4F (20- and 10-ft contraction joint spacing, respectively), only data on joint width opening from joints near the middles of these 2,700-ft sections were used, in order to minimize the effect of the expansion joints at the section ends. The points in Figure 16 were obtained for four temperatures from lines of regression for the various test sections. It should be noted that at 0 deg, joint width increased almost directly in proportion to increased slab length, but at warmer temperatures the rate of joint width increase was not as rapid with increased slab length.

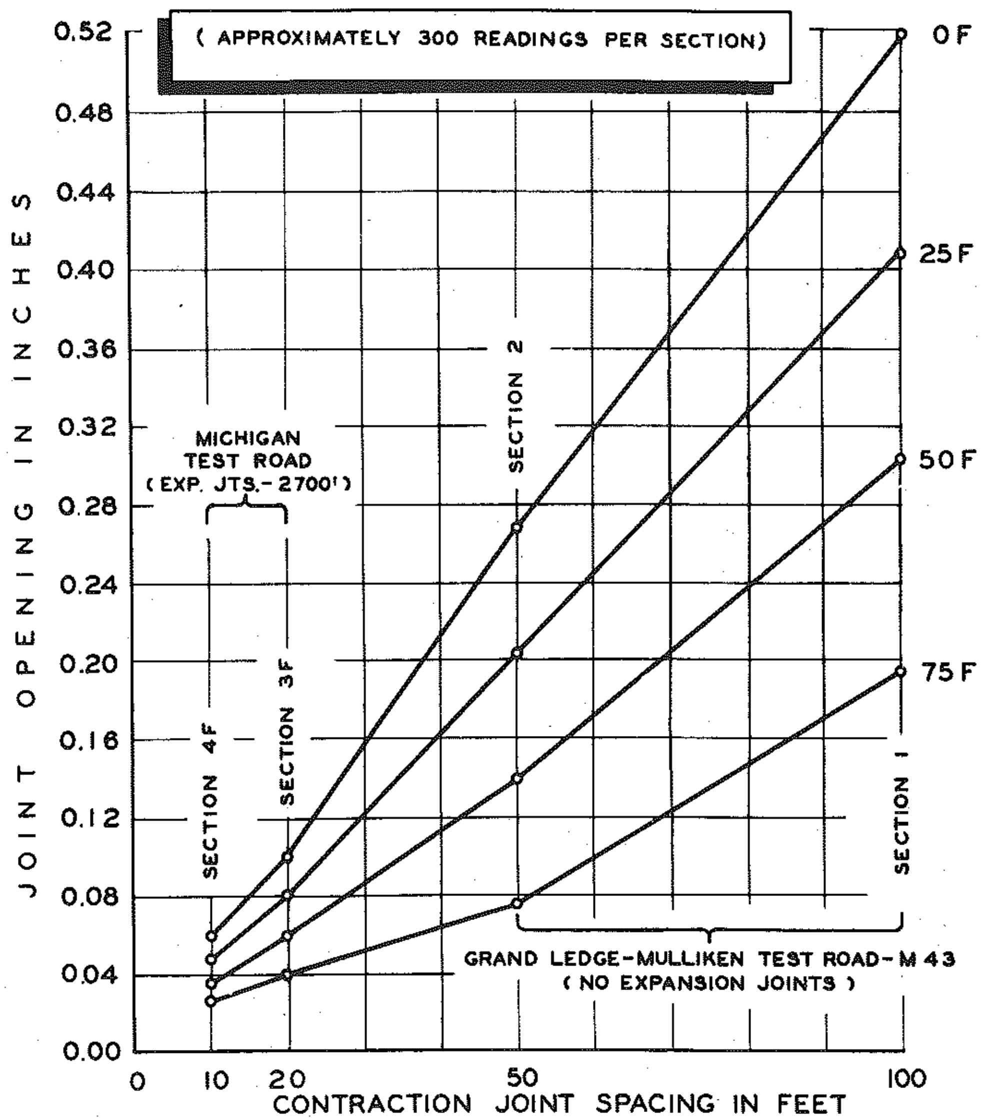


Figure 16.
Effect of Slab Length on Contraction Joint Opening

Frequency Distribution of Joint Widths. In Figures 17 and 18, the frequency distribution of individual joint width measurements is shown for Sections 4F (contraction joint spacing of 10 ft, expansion joint spacing of 2,700 ft) and 3F (20 and 2,700 ft), for joints near the ends and in the middles, under winter and summer temperature conditions. The joint width openings are adjusted to 25 deg in winter and 75 deg in summer, using daily joint width readings for these sections. The general progressive increase in joint width opening is also shown in these figures by plotting measurements for 1942-44, 1948-50, and 1954-56. The mean opening increased 0.018 and 0.044 between 1942-44 and 1954-56, for Sections 4F and 3F, respectively. After 15 years, a winter joint width value of approximately 0.08 in Section 4F and 0.12 in Section 3F was exceeded by 10 percent of the joints at the middles of the sections. Corresponding values for joints near section ends were 41 percent greater for Section 4F and 58 percent greater for Section 3F.

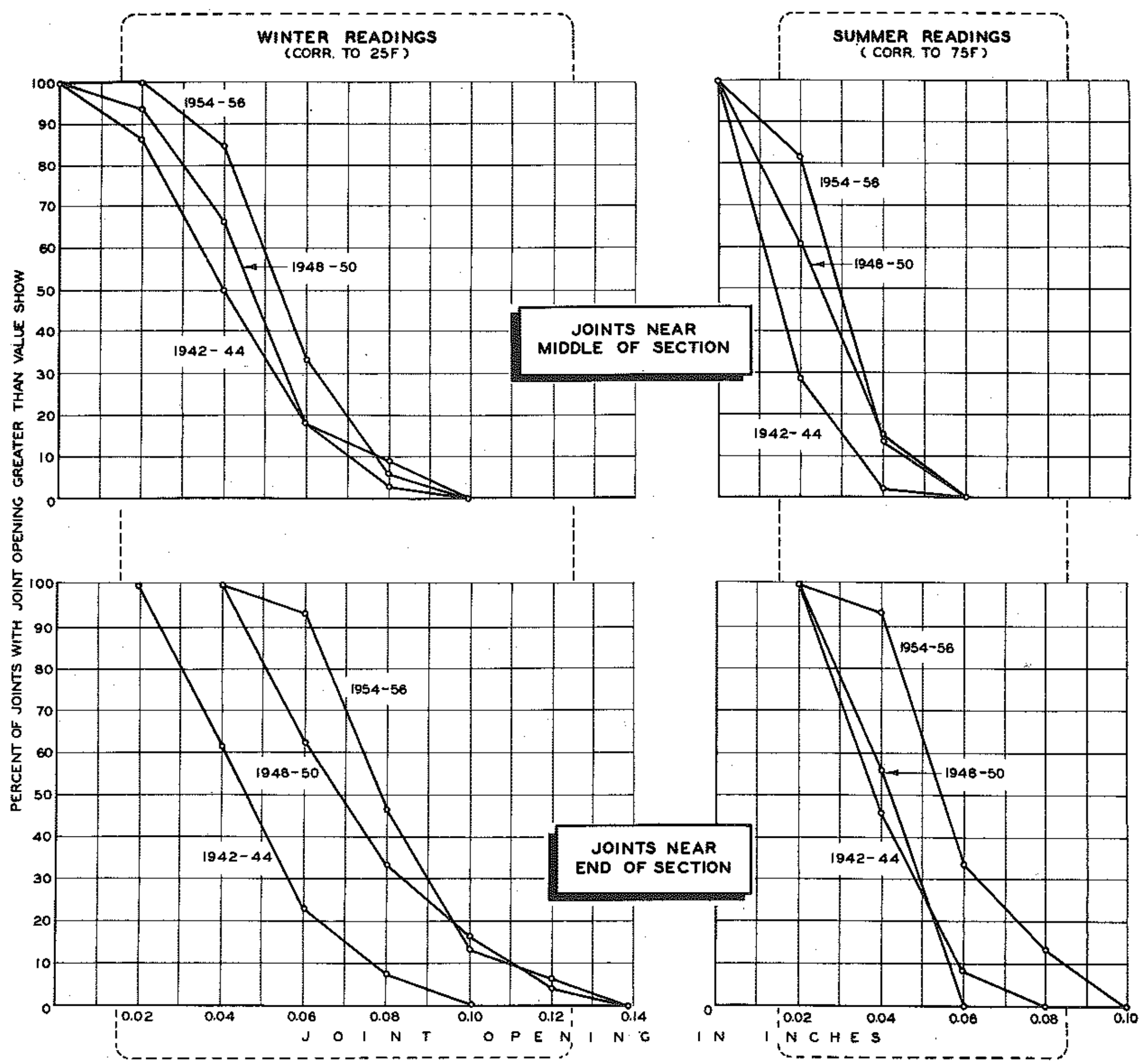


Figure 17. Frequency Distribution of Contraction Joints With Various Openings: Section 4F

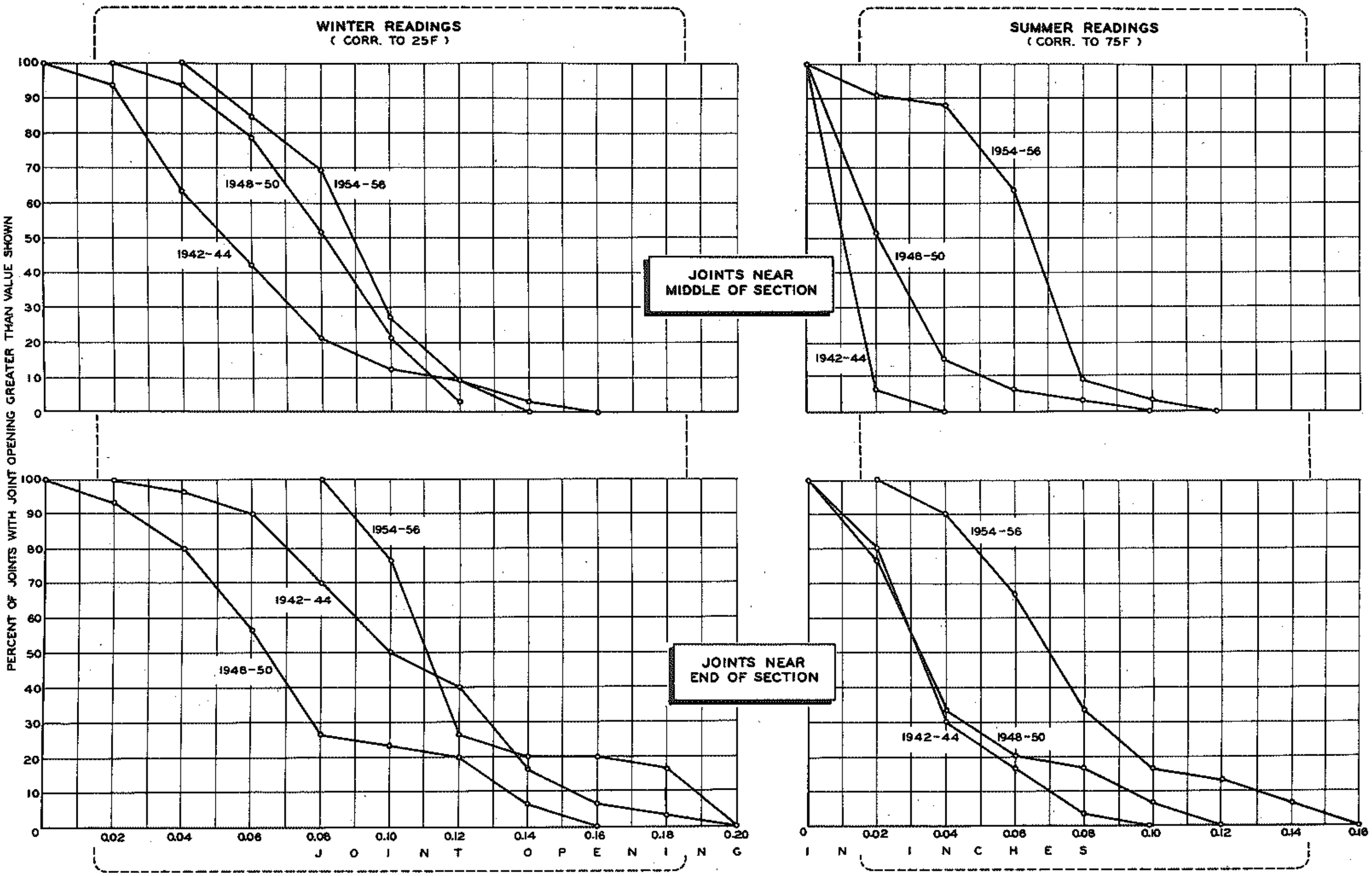


Figure 18. Frequency Distribution of Contraction Joints With Various Openings: Section 3F

Hinge or Warping Joints

In Series 1 and 2, mesh reinforcement of 60 and 37 lb per 100 sq ft respectively was laid continuously through the warping joints. Seasonal joint width measurements were taken at several locations in these series to study the effect of the amount of reinforcing steel on behavior of these joints. The data disclosed that in all cases the joint width increased progressively through the 17-year period. This is shown in Table 6, where average joint openings are given for several sections during 1945, 1950, and 1955, at 25 and 75 deg.

TABLE 6
AVERAGE JOINT OPENING
OF INTERMEDIATE WARPING JOINTS

Series	Section	Steel lb/100 sq ft	Joint Spacing--Feet			Average Joint Opening - Inches					
						Winter (25 F)			Summer (75 F)		
			Expans	Contr	Warping	1945	1950	1955	1945	1950	1955
1	B	60	240	60	30	0.020	0.027	0.048	0.016	0.027	0.039
1	F*	60	2700	60	30	0.030	0.046	0.190	0.026	0.037	0.120
1	F**	60	2700	60	30	0.026	0.041	0.065	0.015	0.027	0.048
2	B	37	240	30	15	0.020	0.087	0.122	0.020	0.039	0.078
2	E*	37	1800	30	15	0.022	0.067	0.112	0.020	0.028	0.063
2	F**	37	2700	30	15	0.027	0.039	0.090	0.017	0.026	0.038

* Joints near end of section. ** Joints near middle of section.

Progressive increases in joint width were greater near the ends of the longer sections 1F and 2E, where width increased approximately 530 and 410 percent respectively, between 1945 and 1955. It may be noted that joints in Sections 1B and 1F (60 lb of steel per 100 sq ft) opened less than those in Sections 2B and 2F (37 lb of steel per 100 sq ft), even though the former were spaced twice as far apart. In all of Series 2 (A through F), contraction joint widths differed very little from summer to winter after about the twelfth year. Undoubtedly, reinforcing steel had ruptured and all or most of the pavement movement was taking place at the warping joints. For example, at the middle of Section 2F, average opening of these joints during the final winter was 0.005 in. greater than for corresponding contraction joints, and the warping joints were moving approximately three times as much as the contraction joints with seasonal temperature change.

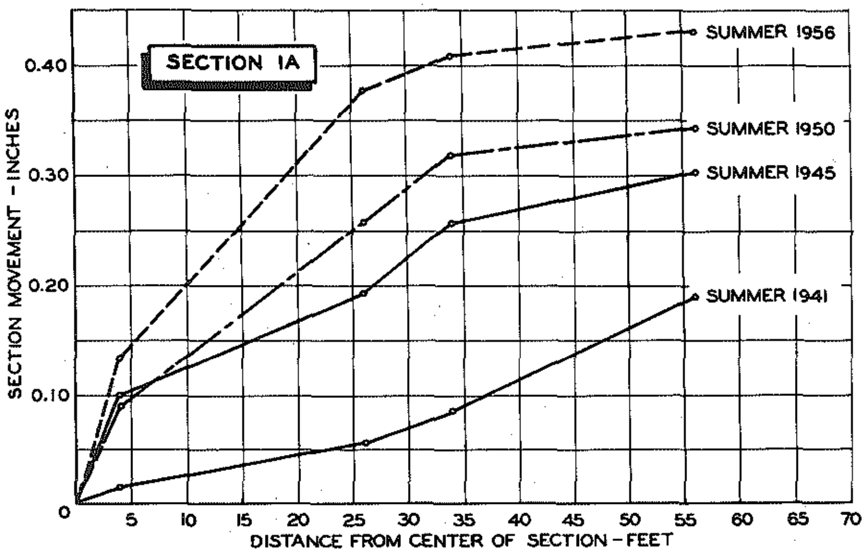
The decrease in seasonal contraction joint movement with age was not as apparent in Series 1; instead, widths at given temperatures remained relatively constant throughout the 17 years. At the middle of Section 1F, the average opening of warping joints was about half as great as for corresponding contraction joints, and warping joints were moving only about half as much from summer to winter as were contraction joints.

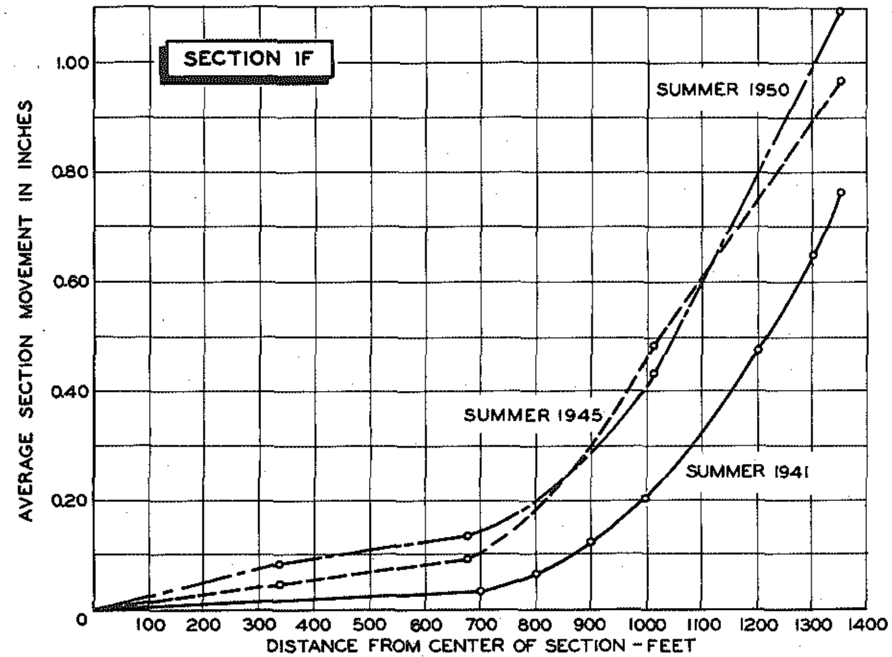
Daily Changes in Joint Width

In conjunction with the seasonal joint width measurements, certain joints were selected for daily observations. Readings were taken on the same joints early in the morning while pavement was cool and in the afternoon when the pavement would normally be at its maximum temperature. Relationships for daily joint width movements for all series have been expressed in comparable terms, such as change in joint width by in. per deg, versus length of section and joint spacing. Complete information on daily joint width movement will be found in Highway Research Board Report No. 17-B (1956). In 1948, daily readings were discontinued as a part of routine observations.

Pavement Movement

In certain sections of Series 1, 2, 3, and 4, reference monuments were established to measure 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 the ends and midpoints of Sections 3A, 4A, 1C, 4C, 1D, 3D, 2F, and 3F. Figure 19 shows the relationship between pavement movement and distance from the centers for Sections 1A, 1F, and 4F. Time's effect is illustrated by the progressive increase in amount of section movement from the first year to the last. Most of the sections were instrumented for measurement of pavement movement only at the center and ends. However, by comparing end movements of the various sections, it is possible to determine the effects of contraction, expansion, and intermediate warping joint spacing on the magnitude of this movement (Figure 20).





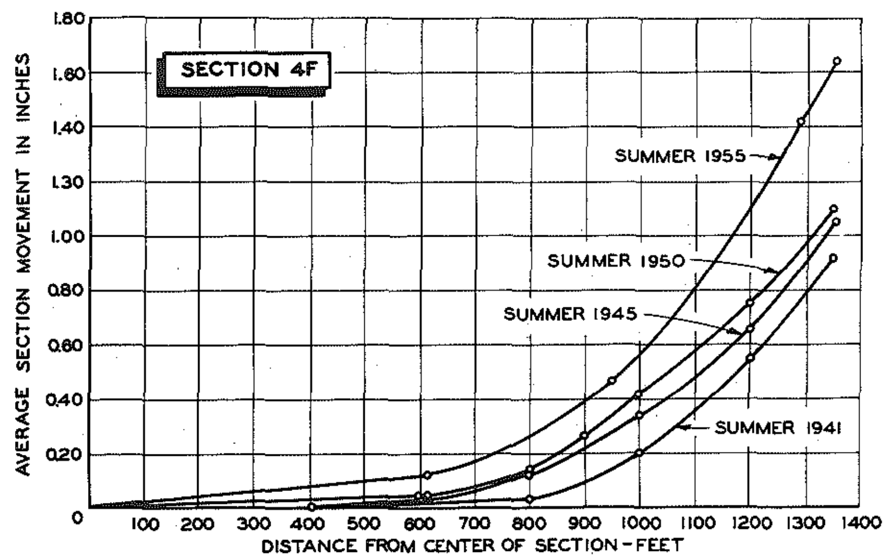


Figure 19. Relation Between Section Movement and Distance from Center of Section

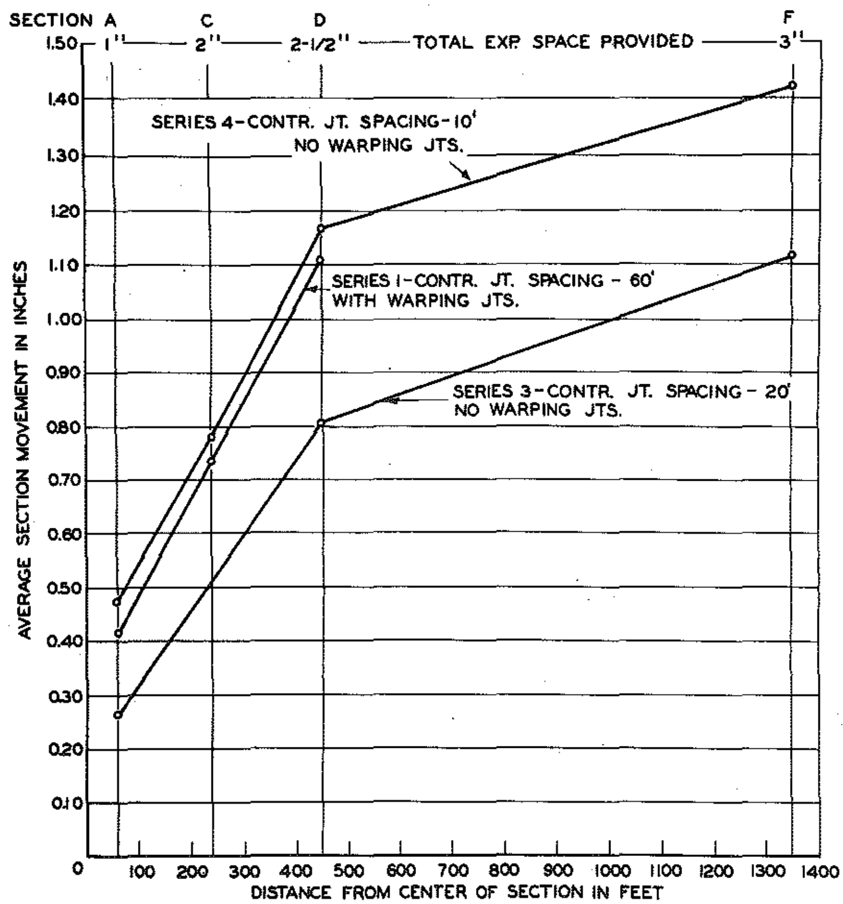


Figure 20. Effect of Expansion and Contraction Joint Spacing on Magnitude of End Movement: 1955

Pavement Performance in Relation to Joint Spacing

Physical behavior of the pavement with respect to different slab lengths and varying expansion joint spacing was evaluated in relation to cracking, spalling, and roughness.

Cracking. The linear feet of pavement cracking occurring in Series 1 through 4 is summarized in Table 7. These data show that the amount of transverse cracking decreases rapidly as slab length decreases. In 1955, the 10-ft slabs had no transverse cracking and the 15-ft slabs had 83 linear feet, while the 20- and 30-ft slabs had about three and six times more transverse cracking respectively, than the 15-ft slabs.

TABLE 7
PAVEMENT CRACKING AS RELATED TO SLAB LENGTH

Series	Slab Length Ft	Total Length of Series, Ft	Pavement Cracking in Feet								Pav't Cracking 1955 ft per mi
			Transverse		Diagonal		Longitudinal		Total		
			1950	1955	1950	1955	1950	1955	1950	1955	
1	30	8,820	253	494	0	12	6	6	259	512	306
3	20	8,820	128	233	10	10	35	35	173	278	166
2	15	8,820	66	83	0	0	0	18	66	101	60
4	10	8,820*	0	0	0	0	0	16	0	16	14

*Part of Series resurfaced in 1953; therefore, 1955 survey was based on only 6,020 feet.

Spalling. A 1950 survey of spalled concrete adjacent to contraction joints indicated that spalling was greatest for 30-ft slabs and decreased almost directly with decreasing slab length. In 1955, the percent of spalled joints had approximately doubled over the 1950 figures, except for Series 3 where the percent of spalled joints had increased almost sixfold (Table 8).

TABLE 8
PAVEMENT SPALLING AT CONTRACTION JOINTS
AS RELATED TO SLAB LENGTH

Series	Slab Length, Ft	Percent of Joints Spalled	
		1950	1955
1	30	28	62
2	15	19	40
3	20	13	72
4	10	5	10

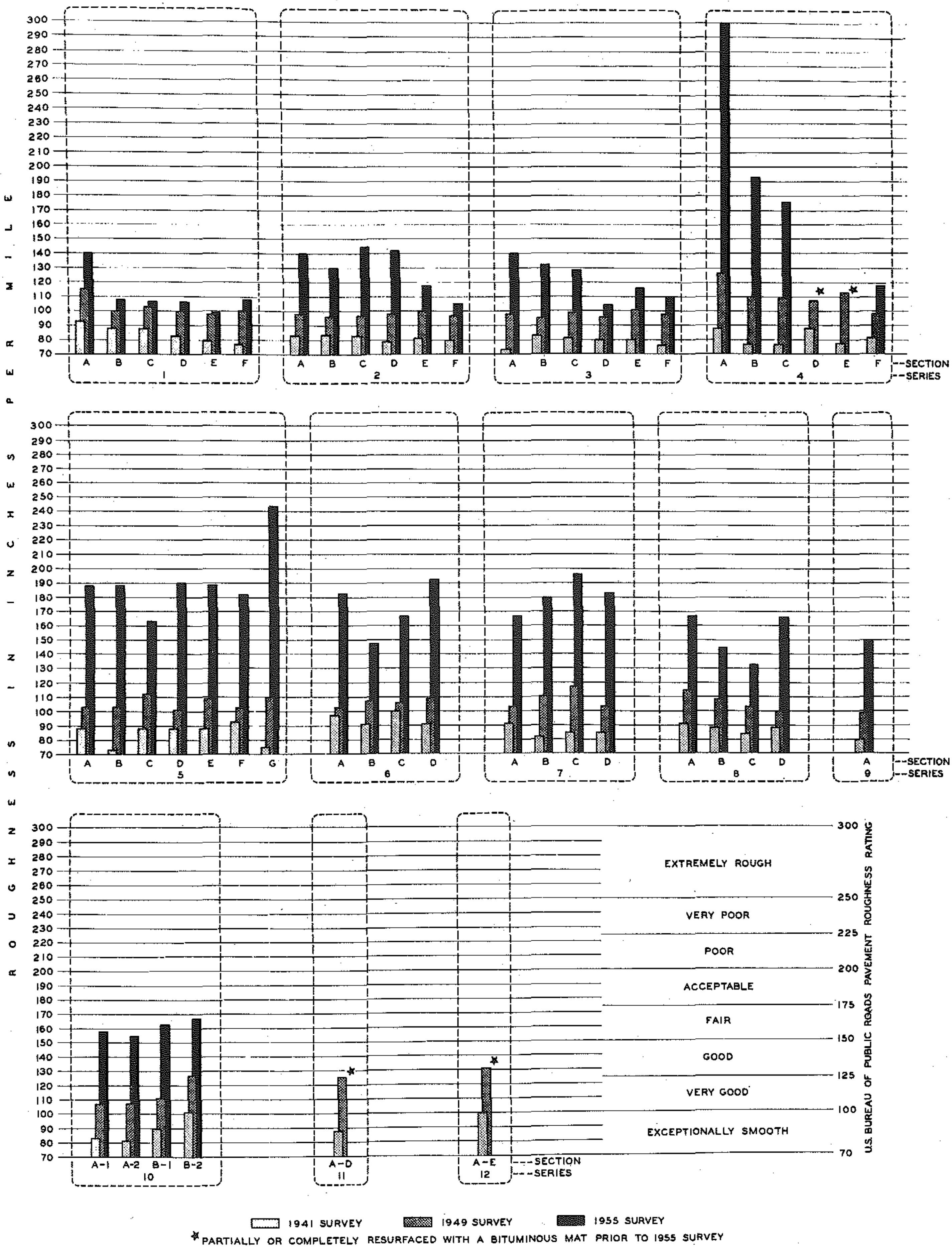


Figure 21. Pavement Roughness

Roughness. Three series of roughness measurements were made for the entire Design Project by Bureau of Public Roads personnel, using their roughometer. The riding qualities of various sections of the pavement were studied, especially where contraction or expansion joint spacing was variable, to compare changes in roughness with time. In conducting the roughness measurements, each section was taken as an increment in order to compare surface roughness conditions in terms of the construction variables for individual sections.

In comparing roughness of Series 1 through 4 shortly after construction, it should be noted that these values were very similar, with a total range from 79 to 85, as shown in Figure 21. After eight years of traffic, roughness had increased about 20 percent for the first three series, but about 34 percent for Series 4. By 1955, roughness had increased by 32, 37, 67, and 132 percent for Series 1, 3, 2, and 4 respectively. This increase was largely due to scaling, especially along transverse joints and the longitudinal joint. By 1955, the percent of the pavement surface having scaling was 1.6, 4.1, 6.2, and 39.9 for Series 1, 3, 2, and 4 respectively, indicating that shorter slabs had the greater amount of scaling. A partial explanation for this is the fact that scaling generally started at the transverse joints and then proceeded to the slab interior, the 10-ft slabs being more vulnerable to scaling because of the exceptional amount of hand finishing required.

Slab Warping Measurements. In 1949 and 1950, measurements were taken of the amount of vertical movement of the corners of certain slabs at contraction joints, along with daily change in pavement temperature from morning to afternoon. This was done for four days at from three to eight joints in each of the four sections, illustrating the effect of slab length on warping movement (Table 9).

General Surface Condition. In general, the concrete surface throughout Series 1 through 4 deteriorated gradually during the 17 years of service, for the most part in the form of spalling at joints and in development of light to heavy scaling. This scaling usually started along both the transverse and longitudinal joints and worked progressively toward the slab centers. Deterioration of this type was more severe in some sections than others, indicating that concrete lacked uniform quality throughout the project.

Scaling progressed rapidly after 1950, and the riding quality of the pavement surface became so bad by 1957 that the entire project had to be resurfaced. Before this, in 1953, parts of Sections 4D and 4F and all of

TABLE 9
SLAB WARPING

Section	Contr Joint Spacing, Ft	Warping Joint Spacing, Ft	Slab Length, Ft	Month	Vertical Movement in 0.001 in. per deg F
1F	60	60	30	Oct	2.10
				June	0.99
2F	30	30	15	Oct	1.74
				June	0.36
3E	20	--	20	Oct	1.76
				June	----
4E	10	--	10	Oct	0.64
				June	----

Section 4E required resurfacing when advanced scaling along joints spaced at 10-ft intervals produced a very rough-riding surface. In Table 10, scaling is tabulated for each section of Series 1 through 4. It may be noted that Series 4 had by far the greatest amount of scaling (39.9 percent) but even there it was not uniform, Section 4F having as little as 1.2 percent.

Variation in pavement condition in Series 1 through 4 is shown pictorially in Figures 22 through 25. The unscaled surface of part of Section 1C is shown in Figure 22, while Figure 23 illustrates light to medium scaling in Section 4C. More advanced scaling is illustrated in Figure 24, and typical scaling along a transverse joint in Figure 25.

One blowup occurred in Series 1 through 4, at a construction joint in Section 1F in 1954, and is illustrated in Figure 26.

A common site of pavement deterioration was at construction joints. A typical example is shown in Figure 27. Twelve of 26 construction joints in Series 1 through 4 had spalling or extensive deterioration, in every case on concrete placed at the end of a day's pour.

TABLE 10
TABULATION OF PAVEMENT SCALING - 1955

Series	Section	Percent of Pavement Surface Scaled			
		Light Scale	Medium Scale	Heavy Scale	Total Scale
1	A	0.0	0.4	5.6	6.0
	B	0.6	1.2	0.4	2.2
	C	0.1	0.3	0.4	0.8
	D	0.0	0.0	0.0	0.0
	E	0.0	0.0	0.0	0.0
	F	0.0	0.4	0.2	0.6
Average-Series 1		0.1	0.4	1.1	1.6
2	A	0.2	3.2	0.3	3.7
	B	0.1	2.8	2.2	5.1
	C	0.0	10.3	4.5	14.8
	D	0.6	4.6	3.4	8.6
	E	0.0	3.4	1.4	4.8
	F	0.0	0.1	0.1	0.2
Average-Series 2		0.1	4.1	2.0	6.2
3	A	0.0	1.7	1.2	2.9
	B	0.1	3.1	0.5	3.7
	C	0.0	3.0	3.9	6.9
	D	2.2	0.6	2.8	5.6
	E	0.8	2.7	0.0	3.5
	F	0.1	1.8	0.0	1.9
Average-Series 3		0.5	2.2	1.4	4.1
4	A	0.0	0.0	59.0	59.0
	B	0.0	0.0	54.0	54.0
	C	0.0	33.0	0.0	33.0
	D	0.0	0.7	52.0	52.7
	E		(Resurfaced in 1953)		
	F	0.0	1.0	0.2	1.2
Average-Series 4		0.0	6.9	33.0	39.9

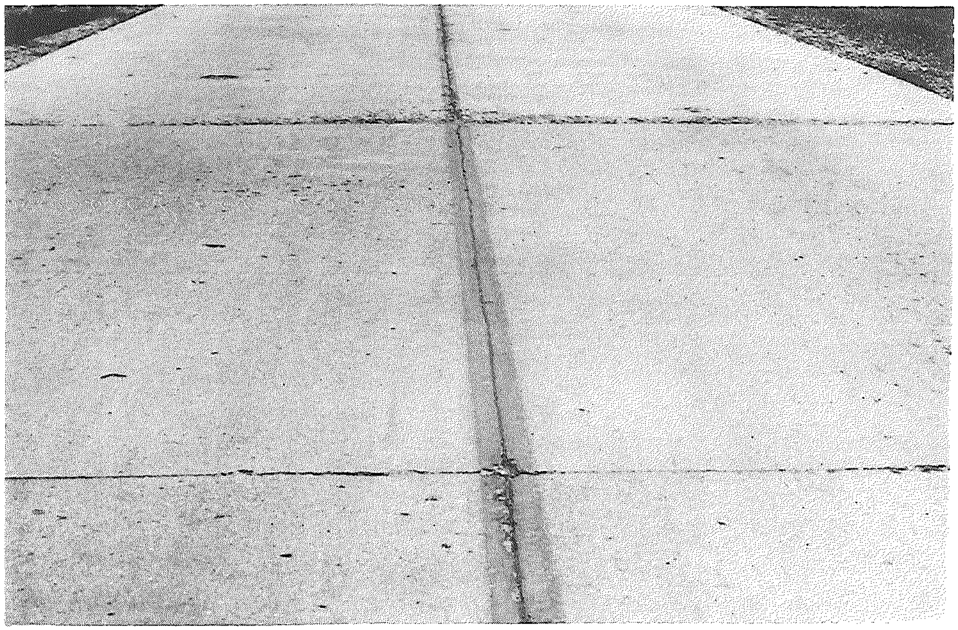


Figure 22. Good Condition of Pavement Surface, Section 1C


Figure 23. Medium Scale of Pavement Surface, Section 4C


Figure 24. Heavy Scale of Pavement Surface, Section 4E

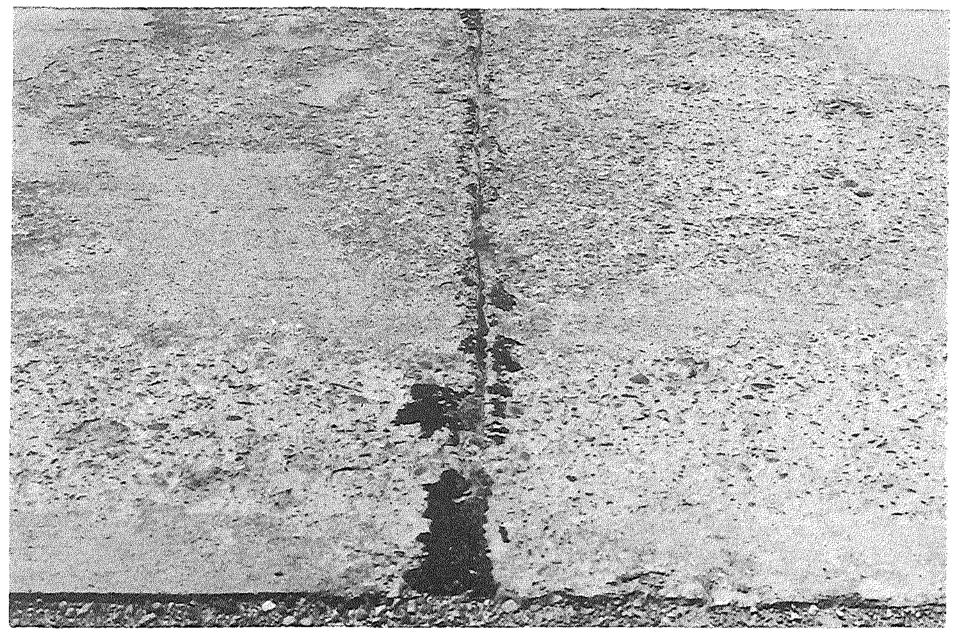


Figure 25. Typical Scaling Along Transverse Joint, Section 4E

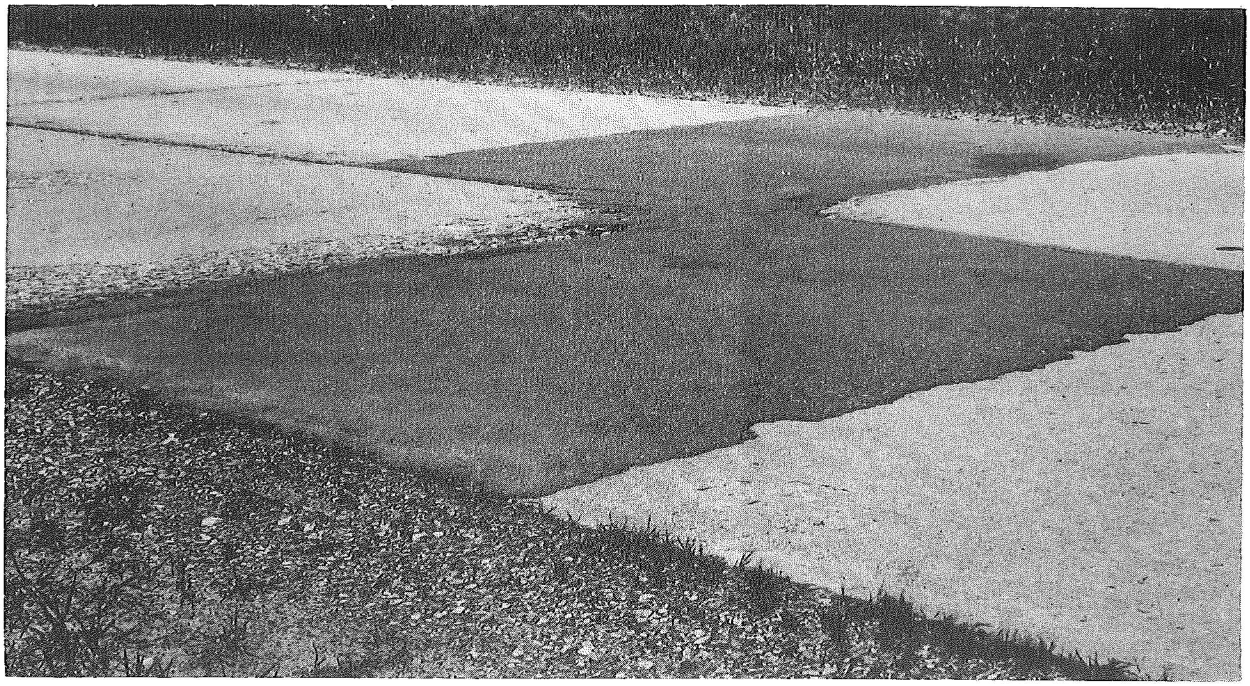


Figure 26.
Pavement Blowup
at Construction Joint,
(Station 841+00)
Section 1F. -1954

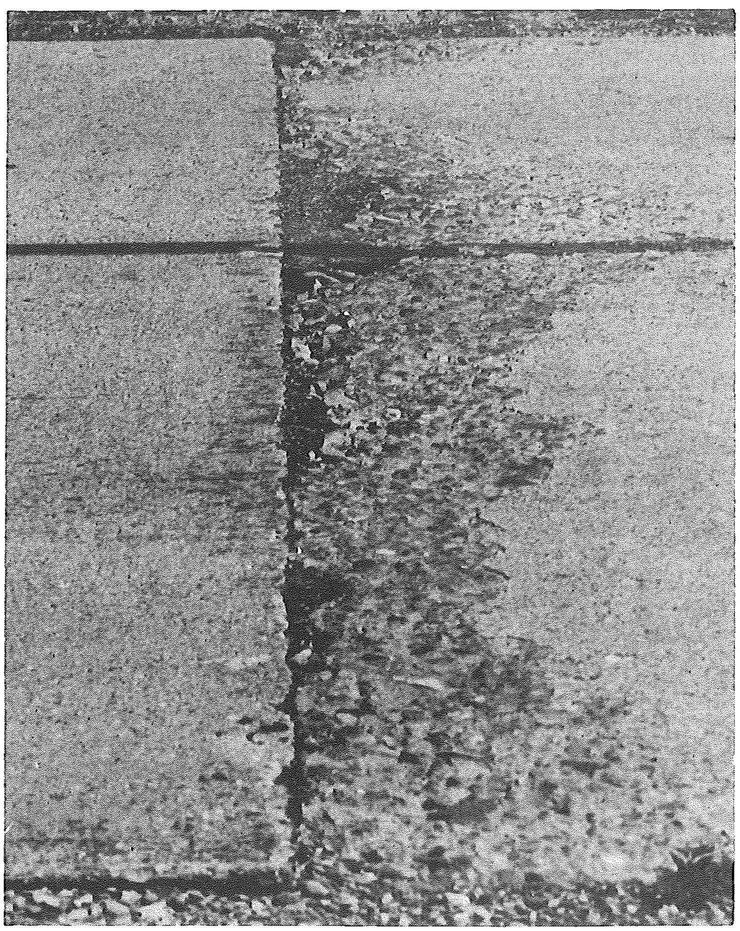


Figure 27.
Typical Pavement
Deterioration
At Construction Joint,
(Station 789+32)
Section 1C.

Summary

The following significant facts are apparent from the joint width study:

1. Contraction joint width movements were materially affected by the combined width and spacing of expansion joints. For contraction joint spacings of 30 ft or less, joint width movements were affected by expansion joint spacings up to about 900 ft. The data indicated that for contraction joint spacings greater than 30 ft, the effect of expansion joint spacing dropped from 900 ft to about 400 ft.

2. For 10- to 100-ft contraction joint spacings without expansion joints (or for joints removed by distance from the effect of expansion joints), contraction joint width at winter temperatures of 0 deg increased approximately in proportion to increase in contraction joint spacing. However, as temperature increased, this proportion decreased until at high summer temperatures joint width did not change notably regardless of slab length.

3. Individual contraction joint width measurements were found to vary considerably in all sections. This would indicate that in plain concrete pavement design, the frequency distribution of joint widths for winter conditions should be considered rather than the mean joint width values.

4. All contraction joints acquired a permanent opening, gradually increasing in 10 to 15 years to a significant value which under certain conditions might materially affect joint performance.

TABLE 11
PAVEMENT CRACKING AND SPALLING AS RELATED TO SLAB LENGTH THICKNESS

Section	Slab Length, Ft	Slab Thickness, In.	Total Length of Series, Ft	Pavement Cracking in Feet								Pav't Cracking 1955 ft per mi	Percent of Contraction Jts Spalled	
				Transverse		Diagonal		Longitudinal		Total				
				1950	1955	1950	1955	1950	1955	1950	1955		1950	1955
6A	30	8 uniform	240*	139	170	0	0	0	0	139	170	3740	30	30
6B	20	8 uniform	334*	48	60	0	8	0	0	48	68	1070	43	68
6C	15	8 uniform	326*	0	0	0	0	0	0	0	0	0	61	85
6D	10	8 uniform	600	0	0	0	0	0	0	0	0	0	25	34
Total - Series 6			1500	187	230	0	8	0	0	187	238	838	40	54
7A	30	8-6-8	600	146	161	0	0	0	0	146	161	1420	10	20
7B	15	8-6-8	600	0	0	0	0	0	0	0	0	0	5	36
7C	20	8-6-8	600	110	110	0	0	0	0	110	110	970	7	64
7D	10	8-6-8	600	22	22	0	0	0	0	22	22	194	0	9
Total - Series 7			2400	278	293	0	0	0	0	278	293	645	5	32
8A	30	7 uniform	360	22	22	23	23	0	0	45	45	660	41	79
8B	20	7 uniform	840	11	11	0	0	0	20	11	31	195	24	63
8C	15	7 uniform	600	0	0	3	3	0	0	3	3	26	25	38
8D	10	7 uniform	600	0	0	0	0	0	0	0	0	0	12	24
Total - Series 8			2400	33	33	26	26	0	20	59	79	174	26	51

* Frost heave areas removed from analysis.

In planning the Michigan Test Road, Series 5 through 12 were included to study various factors associated with concrete pavement design, such as thickness, shape of cross section, amount of steel reinforcement, and joint design including load transfer.

Thickness and Cross Section

In the Design Project, four pavement thicknesses including two types of pavement cross section were constructed to study such factors as subgrade load capacity versus slab thickness, and the value of balanced or thickened-edge cross section versus uniform cross section. The following four pavement thicknesses were used: 9-7-9 in. (Series 1-4), 8-6-8 in. (Series 7), 7-in.uniform (Series 8), and 8-in. uniform (Series 6).

Contraction Joint Width. Using certain sections of Series 6, 7, and 8 and sections in Series 1 through 4, it was possible to compare contraction joint widths for the four thicknesses on the basis of common contraction and expansion joint spacing. In Series 6, 7, and 8, contraction joint spacings of 10, 15, 20, and 30 ft were used with 120- and 300-ft expansion joint spacing. In Figure 28, joint widths for four temperatures are

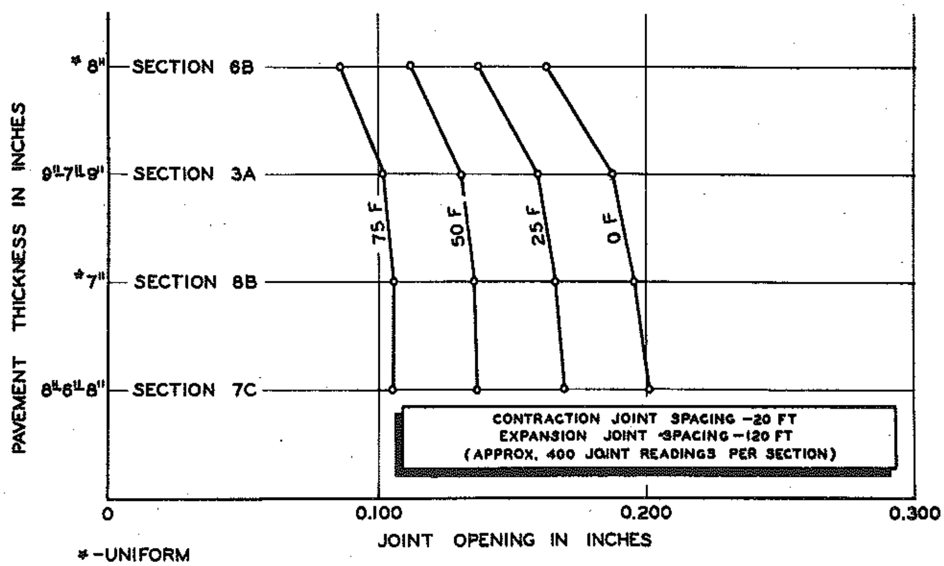


Figure 28. Effect of Pavement Thickness on Contraction Joint Opening

shown for four pavement thicknesses with 20-ft contraction and 120-ft expansion joint spacings. This information indicates that amount of opening or seasonal variation in opening was not significantly different for any of the pavement thicknesses or cross sections. In Series 6, 7, and 8, contraction joint movement was noticeably reduced when transverse cracks developed in the slabs which were not reinforced. Instead, movement then took place for the most part in these cracks.

Physical Condition of Pavement. Pavement cracking varied considerably among the various series, from a maximum of 838 ft per mi of pavement for Series 6 to a minimum of 174 ft per mi for Series 8 (Table 11). Frost heave areas in Series 6 are not included in this Table, and were fully discussed in the 10-year report on the Design Project. The pavement cracking history of Series 6, 7, and 8 is illustrated in Figures 29, 30, and 31. Series 8 was constructed entirely on excellent granular subgrade soil, while Series 6 and 7--except for Section 7D--were placed on a subbase over questionable subgrade material.

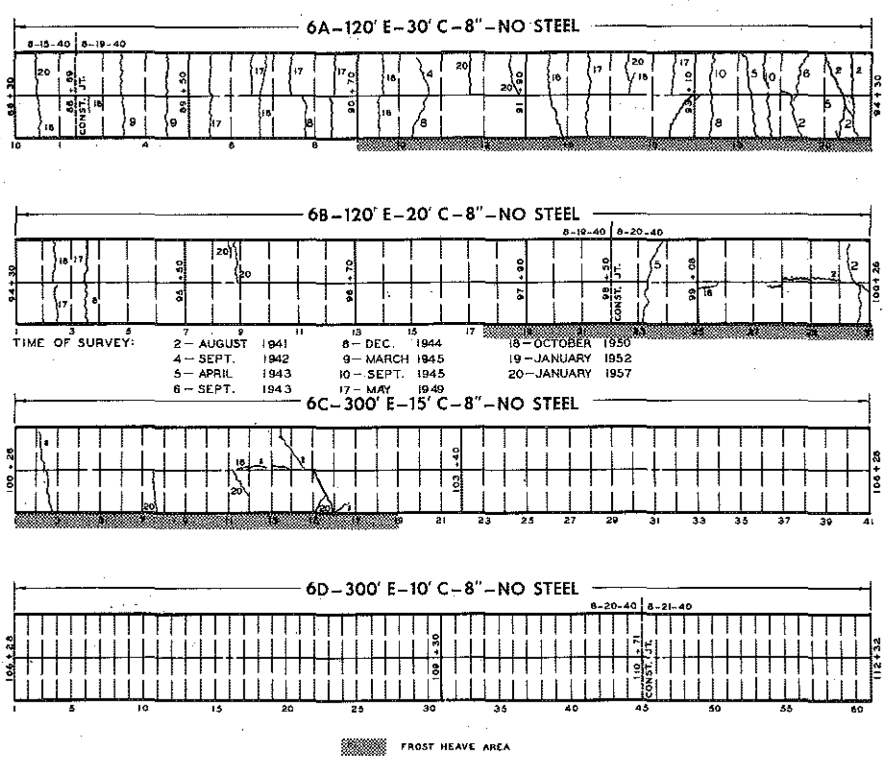


Figure 29. Pavement Condition: Series 6

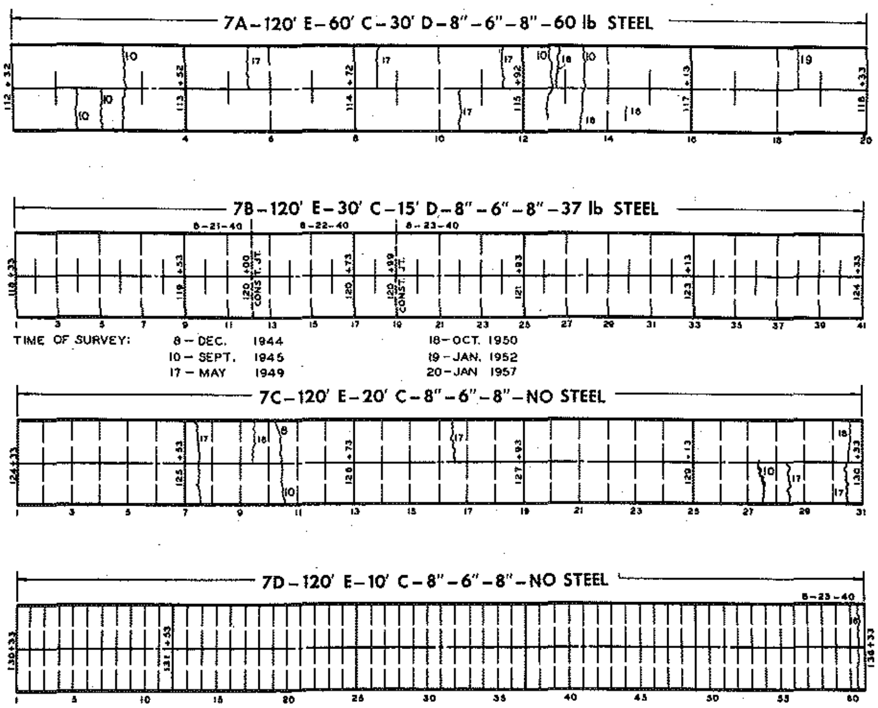


Figure 30. Pavement Condition: Series 7

The cracking disparity between Series 6, 7, and 8 (Table 11) cannot be ascribed to differences in cross section, but rather to accidental variations in subgrade support, concrete quality, or both. This indicates the very rigid control required in subgrade preparation and all other phases of experimental highway construction, to insure that pavement performance depends on the parameter under study rather than some other insufficiently controlled parameter. In spite of these variables, the relationship between cracking and slab length does verify evidence, established previously in Series 1 through 4, that longer slabs have more transverse cracks.

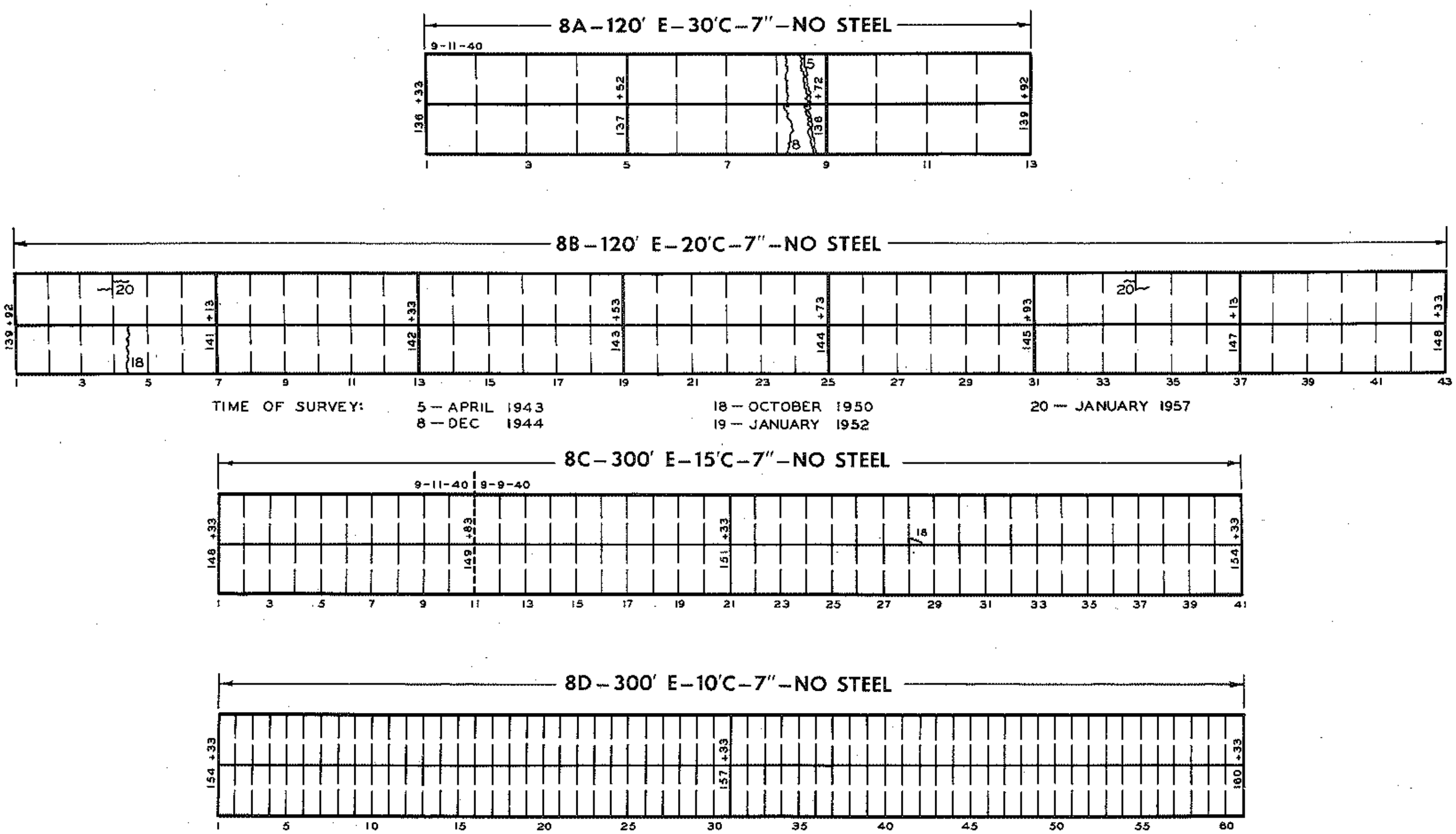


Figure 31. Pavement Condition: Series 8

Shortly after construction, pavement roughness varied from 86 for Series 7, to 95 for Series 6 (see Figure 21). After 15 years of weathering and traffic, the roughness of Series 7 had increased 118 percent, while Series 6 and 8 had increased 83 and 76 percent respectively. The percent increase in roughness was related to the amount of scaling which had occurred. For example, Series 7 with 118 percent increase in roughness also had the greatest percentage of scaling (76 percent), while Series 6 and 8 had only 46 and 27 percent with a corresponding roughness increase of 83 and 76 percent respectively. A comparison of roughness and scaling with slab length for Series 6, 7, and 8, shows average roughness values of 174, 161, 168, and 171, with corresponding scaling percentages of 59, 50, 45, and 46, for slab lengths of 30, 20, 15, and 10 ft respectively.

The general physical condition of the pavement surface at time of resurfacing for each section is shown in Figures 32 and 33.

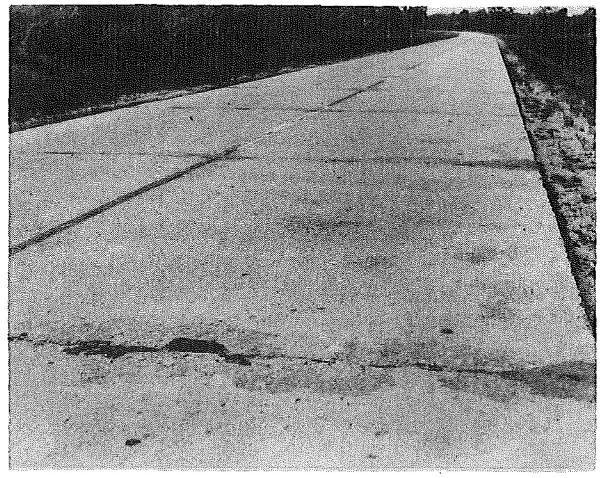
Reinforcement

Two weights of steel reinforcement (60 and 37 lb per 100 sq ft) were used in various sections of the Design Project, while other sections were not reinforced. Both weights were installed in conjunction with warping joints, and the 60-lb reinforcement in continuous slabs of various lengths without intermediate contraction or warping joints.

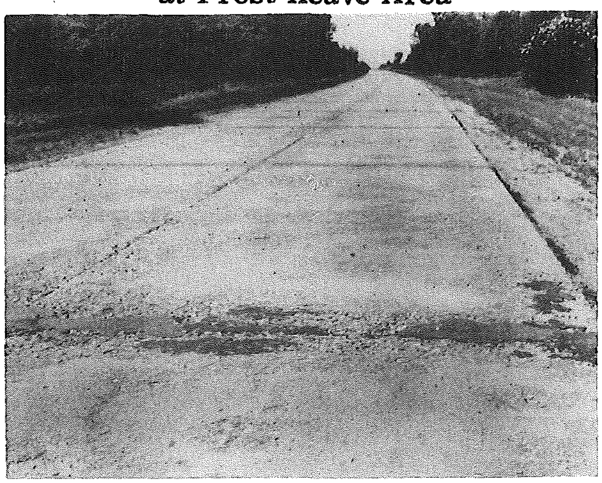
Reinforcement in Relation to Warping Joints. Intermediate warping joints in both Series 1 and 2, with steel reinforcement of 60 and 37 lb per 100 sq ft respectively, widened progressively with age. Comparing Sections 1B and 1F with 2B and 2F, joints with heavier reinforcement opened less even though slab length was twice as great (Table 6). In Series 2, by 1949, three of 27 joints (11 percent) where measurements were taken had widened sufficiently to indicate rupturing of reinforcement. This increased to 50 percent by 1955 and 96 percent by 1957. In contrast, from width measurements of 23 joints in Series 1, it appears that the first break in steel occurred about 1953. By 1955, the joints with broken steel increased to 17 percent, and by 1957 to 30 percent.

Continuous Slabs With and Without Reinforcement. Series 11 and 12 of the Design Project were constructed within the Test Road's Durability Project (Table 1). Steel reinforcement of 60 lb per 100 sq ft was used in Series 11 for continuous slabs of 90, 120, 360, and 600 ft. In Series 12, the same slab lengths were constructed without reinforcement. Over the 17 years, no cracking occurred in Sections A and B of either Series 11 or 12 (slab lengths of 90 and 120 ft). However, the longer sections of both

Section 6A
Station 94+25 Looking South
at Frost Heave Area



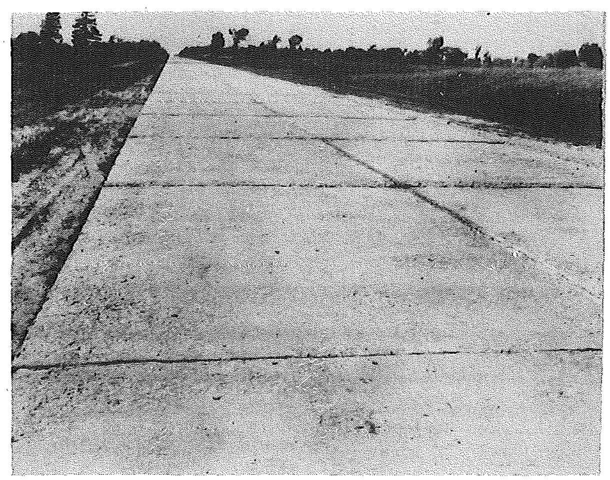
Section 6B
Station 96+00 Looking North



Section 7A
Station 114+00 Looking North

Section 7B
Station 124+00 Looking South

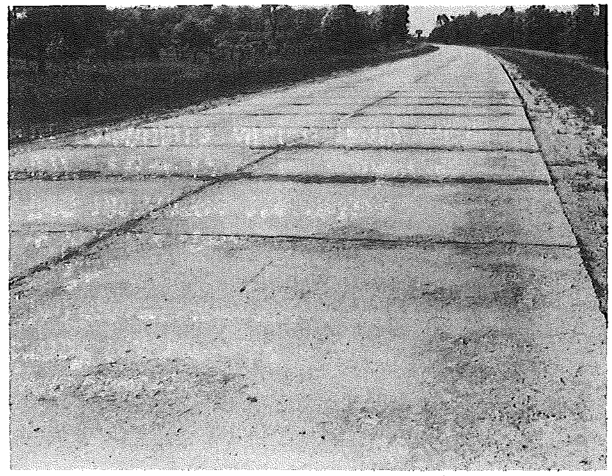
Section 8A
Station 138+30 Looking South



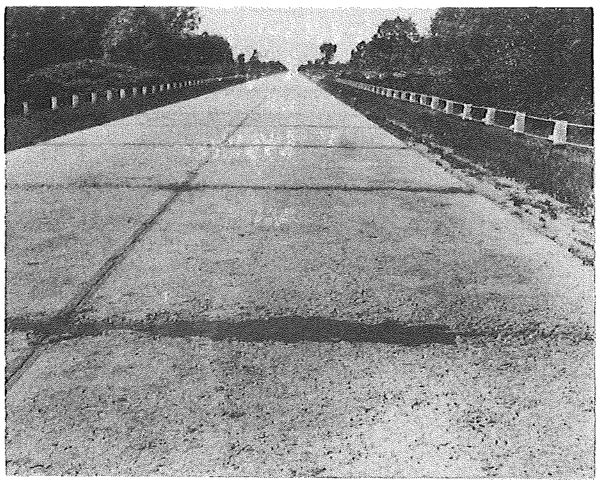
Section 8B
Station 141+00 Looking North

Figure 32. General Views of Pavement Sections A and B of Series 6, 7 and 8

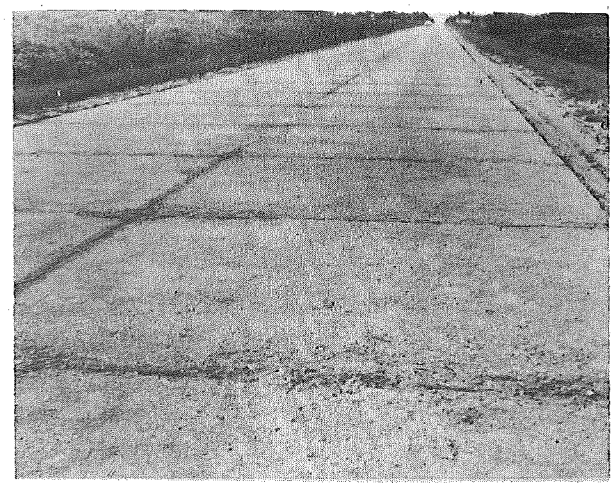
Section 6C
Station 106+00 Looking South



Section 6D
Station 106+30 Looking North

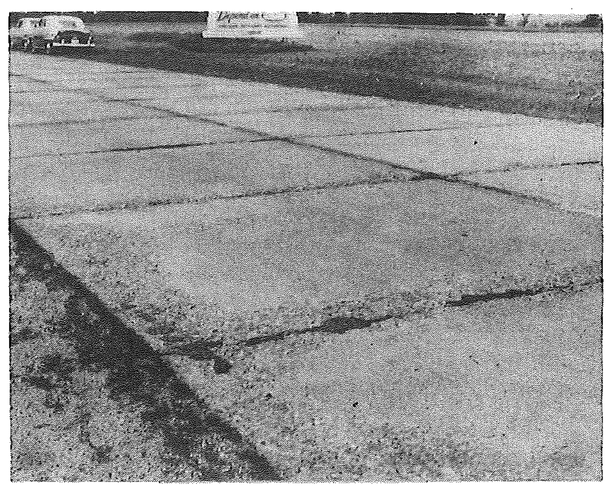


Section 7C
Station 124+50 Looking North



Section 7D
Station 130+40 Looking North

Section 8C
Station 150+00 Looking North



Section 8D
Station 158+00 Looking North

Figure 33. General Views of Pavement Sections C and D of Series 6, 7 and 8

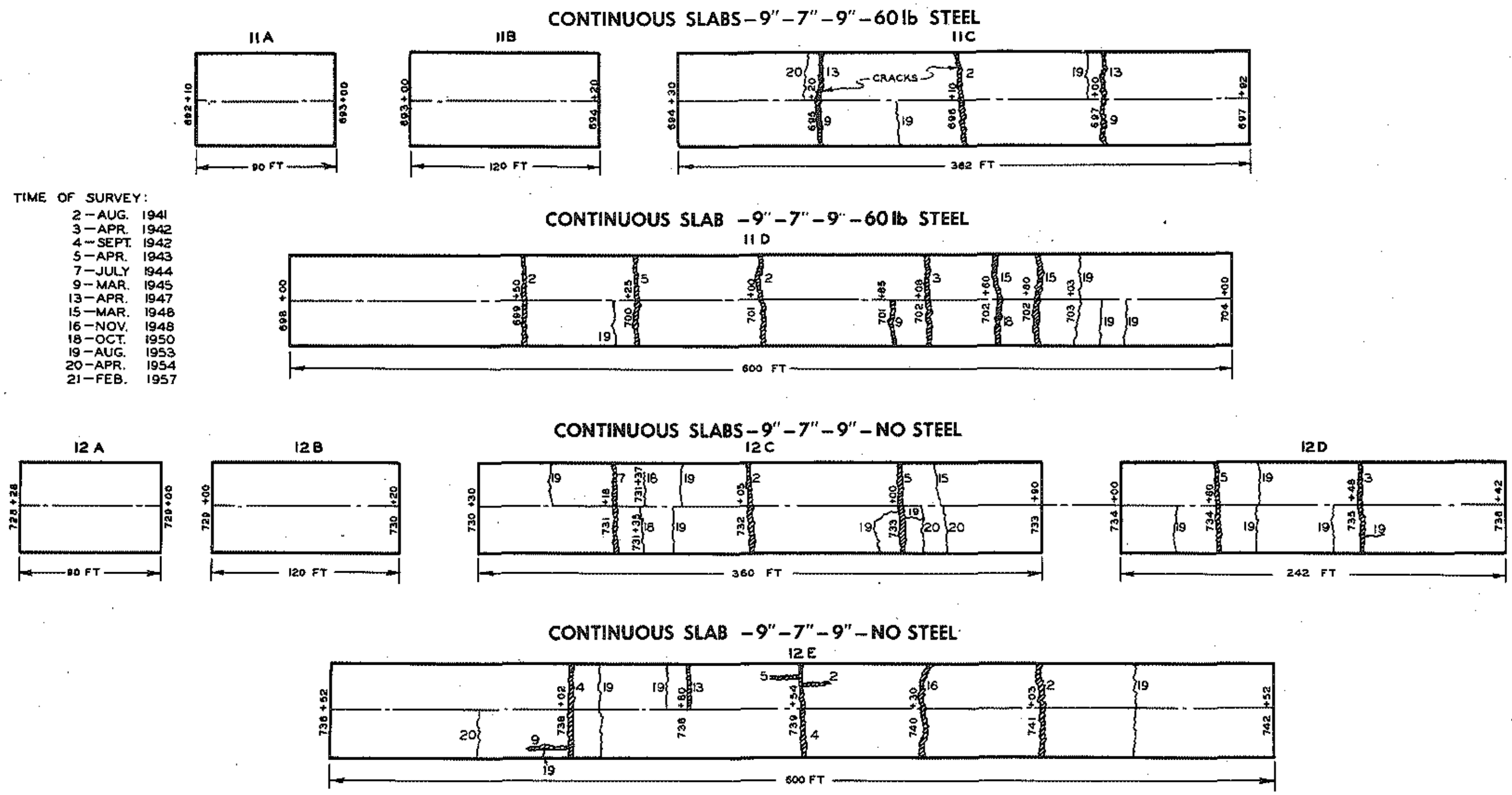


Figure 34. Pavement Condition: Series 11 and 12

series had numerous transverse cracks which opened appreciably (Figure 34). These cracks were instrumented and the openings measured twice a year. In Section 11C, these readings showed that the reinforcing steel had broken at Sta 696+10 as early as 1947, but at Sta 695+20 and 697+00 the steel apparently remained intact until 1949. A tabulation of pavement cracking is given in Table 12. The amount of transverse cracking was not significantly different in the two series. However, the reinforced pavement was in better general physical condition than the nonreinforced.

In May 1957, load deflection measurements were made on certain cracks in these series to determine the amount of load transfer taking place. For the cracks tested, the average opening was 0.15 in. and the average load transfer value was 21 percent. At the same time, certain cracks were measured in Series 1 and 2, where the steel was unbroken, and the average opening was about 0.05 in. At these cracks, the load transfer value averaged 46 percent, where 50 percent would indicate a perfect rating.

TABLE 12
PAVEMENT CRACKING IN CONTINUOUS SLABS
Series 11 and 12

Section	Slab Length Ft	Steel Reinf lb/100 sq ft	Pavement Cracking in Feet								Pav't Cracking 1957 ft per mi
			Transverse		Diagonal		Longitudinal		Total		
			1950	1957	1950	1957	1950	1957	1950	1957	
11A	90	60	0	0	0	0	0	0	0	0	0
11B	120	60	0	0	0	0	0	0	0	0	0
11C	362	60	66	99	0	0	0	0	66	99	1440
11D	600	60	164	208	0	0	0	0	164	208	1830
Total - Series 11			230	307	0	0	0	0	230	307	-
12A	90	0	0	0	0	0	0	0	0	0	0
12B	120	0	0	0	0	0	0	0	0	0	0
12C	360	0	66	173	0	0	0	20	66	193	2830
12D	242	0	44	88	0	0	0	0	44	88	1920
12E	600	0	99	168	0	0	62	62	161	230	2020
Total - Series 12			209	429	0	0	62	82	271	511	-

Shortly after construction, the roughness values for Series 11 and 12 were 97 and 93, respectively (see Figure 21). After eight years these roughness values increased 5 percent for Series 11, and 11 percent for Series 12. By 1955, the percent increase was 34 and 83 for Series 11 and 12 respectively. The variations in roughness by 1955 of 130 for Series 11 and 170 for Series 12 cannot be ascribed to pavement design features, but rather to differences in surface scaling--3 percent for Series 11 compared to 20 percent for Series 12.

Joint Design

For comparative study, the Design Project included several types of expansion and contraction joint designs in use or under consideration when the project was constructed. The joint design study was reported in the 10-year report appearing in Highway Research Board Report No. 17-B (1956). Therefore, only new data of significance in relation to the objectives of this study will be covered here. This includes additional information on mechanical load transfer versus aggregate interlock.

Load Transfer. In May 1957, prior to resurfacing the pavement with bituminous concrete, a series of six load transfer measurements were made at each of nine contraction joints in Section 3D and ten in 3E. Table 13 compares the load transfer rating of Section 3D with dowelled joints to that of Section 3E with only aggregate interlock to effect load transfer across the joints. Although the section ratings differed very little, the dowelled joints were better by 2.6 percent. Tests conducted at a colder temperature probably would have shown a more marked difference between these sections. Unpublished results from previous testing conducted during late fall on the aggregate-interlock type of joints in Section 3E, showed a reduction in load transfer rating to 36 percent for an average joint width opening of 0.064 in. Comparing joint openings with load transfer ratings for individual joints, however, did not show any well-established correlation between these variables for the range of joint width openings encountered during the May 1957 tests.

Measurements of faulting across contraction joints for Section 3D, 3E, 4D, and 4E were made during 1944, 1949, and 1955, to determine the effects of traffic and of slab length on faulting for joints with and without dowels (Table 14). The faulting increased considerably in Sections 3D and 3E (20-ft slabs) from 1949 to 1955, but the percent of joints faulted was about six times greater for the aggregate interlock type of load transfer (Section 3E) than for the dowelled joints of Section 3D.

TABLE 13
LOAD TRANSFER AT CONTRACTION JOINT

Section	Load Transfer Feature	Avg Joint Width Opening, Inches	Avg Load Transfer Rating-Percent[a]
3D	Dowels-3/4 in. x 15 in. at 15 in. spacing	0.063	48.8[b]
3E	Aggregate Interlock	0.045	46.2[b]
3E	Aggregate Interlock	0.064	36.1[c]

(a) Load Transfer Rating Percent =

$$\frac{\text{Defl Unloaded Side of Joint}}{\text{Defl Loaded Side + Defl Unloaded Side}} \times 100$$

(b) Measurements in late spring.
(c) Measurements in late fall.

In Sections 10A and 10B, with 120-ft expansion joint spacing, 15- and 20-ft contraction joint spacings were used. Dowels were installed in all joints of Section 10A and omitted in 10B. The faulting data in Table 14 clearly shows that mechanical load transfer is necessary for short slab construction when expansion joints are spaced at 120 ft. Further, load transfer is particularly needed at expansion joints and, finally, it is quite apparent that the load design feature (3/4- x 15-in. dowels at 15-in. centers) was inadequate for the load and subbase conditions.

Summary

After 17 years the results indicate no difference in performance between the uniform cross section and the balanced or thickened edge cross section, nor have the results brought out any significant differences in structural performance as related to slab thickness. Obviously, the test road traffic has not been sufficient during the 17-year test period to cause structural failure in even the thinnest, 7-in. uniform section.

The most significant finding from the steel reinforcement study concerns its use in connection with pavement design requiring intermediate warping joints, as in Series 1 and 2. Obviously, the warping joints opened sufficiently to permit surface water to reach the steel reinforcement, thereby accelerating rusting and causing eventual breakage of steel. The time element involved in this action would naturally be related to the amount of steel used. In this case, the 37-lb reinforcement started to break in 1948 after about eight years' service, while with the 60-lb reinforcement, the first breakage appeared in 1953, or after 13 years of service.

Aggregate interlock was not sufficient to prevent faulting of the 20-ft slabs regardless of expansion joint spacing. Under certain conditions, the 3/4-in. by 15-in. dowelling system was inadequate to prevent faulting at contraction or expansion joints.

TABLE 14
FAULTING OF EXPANSION AND CONTRACTION JOINTS WITH AND WITHOUT LOAD TRANSFER DEVICES
Both Lanes Included

Series	Year	No. of Contr. Joints Measured	Contraction Joints - Percent					No. of Exp. Joints Measured	Expansion Joints - Percent					Remarks
			Not Faulted	Faulted 1/8 in.	Faulted 3/16 in.	Faulted 1/4 in.	Faulted Over 1/4 in.		Not Faulted	Faulted 1/8 in.	Faulted 3/16 in.	Faulted 1/4 in.	Faulted Over 1/4 in.	
10 A-1	1944	90	91.1	7.8	0.0	1.1	0.0	20	90.0	10.0	0.0	0.0	0.0	3/4 x 15 in. dowels
	1949	90	85.6	10.0	1.1	1.1	2.2	20	95.0	5.0	0.0	0.0	0.0	with 15 in. spacing.
	1955	27	66.7	18.5	11.1	0.0	3.7	16	93.7	6.3	0.0	0.0	0.0	
	1944	126	96.0	3.2	0.8	0.0	0.0	18	100.0	0.0	0.0	0.0	0.0	3/4 x 15 in. dowels
10 A-2	1949	126	92.0	4.8	2.4	0.0	0.8	0	-	-	-	-	-	with 15 in. spacing
	1955	40	67.5	22.5	5.0	5.0	0.0	12	83.3	16.7	0.0	0.0	0.0	
	1944	90	62.2	25.6	7.8	4.4	0.0	18	55.6	22.2	22.2	0.0	0.0	
10 B-1	1949	90	49.0	28.8	12.2	10.0	0.0	18	49.9	16.7	16.7	16.7	0.0	No dowels
	1955	28	25.1	32.1	21.4	21.4	0.0	12	41.7	16.7	8.3	25.0	8.3	
	1944	126	73.7	17.5	4.0	4.8	0.0	18	44.4	11.1	16.7	27.8	0.0	
10 B-2	1949	126	65.1	21.4	3.2	9.5	0.8	16	49.4	12.5	0.0	31.2	6.9	No dowels
	1955	16	37.4	18.8	37.5	6.3	0.0	8	50.0	12.5	12.5	12.5	12.5	
	1944	176	0.0	0.0	0.0	0.0	0.0	0						3/4 x 15 in. dowels
3 D	1949	176	97.2	2.8	0.0	0.0	0.0							with 15 in. spacing
	1955	174	87.4	12.6	0.0	0.0	0.0							
	1944	178	99.4	0.6	0.0	0.0	0.0							3/4 x 15 in. dowels
4 D	1949	178	96.6	2.8	0.0	0.0	0.0							with 15 in. spacing
	1955	23	82.6	13.1	0.0	4.3	0.0							
	1944	178	91.0	8.4	0.0	0.6	0.0							
3 E	1949	178	77.5	18.0	3.9	0.6	0.0							No Dowels
	1955	168	26.2	50.0	14.3	9.5	0.0							
	1944	358	99.2	0.8	0.0	0.0	0.0							
4 E	1949	358	97.5	2.5	0.0	0.0	0.0							No dowels
	1955	0	-	-	-	-	-							
Joints with Dowels - Avg 1955			76.0	16.7	4.0	2.3	0.9		88.5	11.5	0.0	0.0	0.0	
Joints without Dowels - Avg 1955			29.6	33.6	24.4	12.4	0.0		45.8	14.6	10.4	18.8	10.4	

Note - After a period of years the faulting study on certain joints was discontinued because spalling or scaling at the joint made it impossible to obtain accurate faulting measurements.

INCIDENTAL STUDIES

In addition to the major investigations embodied in the Design Project, several incidental studies were introduced into the research program, pertaining to various construction methods of particular interest to the Department. Practically all the incidental studies were completed at an early date, their results being incorporated into previous reports and utilized in framing the Department's current specifications for concrete pavement construction. However, one of the incidental studies, pertaining to stress curing of concrete, continued until final pavement resurfacing in 1957. Since this test section was fully described in previous reports, only the final observations will be reported here.

Stress Cured Concrete

At the end of 17 years, four of the 18 original 100-ft slabs remained uncracked. The total linear feet of cracking was about 770, or 2,260 linear ft per mi of pavement. As shown in the soil profile in Figure 35, cracking in four of the slabs can be directly attributed to abnormal changes in the subgrade caused by undesirable soil conditions, and not to any factor of weakness in pavement structural performance.

The movement of expansion joints connecting uncracked slabs of stress cured concrete is shown in relation to concrete pavement temperature in Figure 35. For comparison, expansion joint movement is shown for similar uncracked conventional slabs with expansion joints spaced at 120 ft. That expansion joints in conventional slabs became permanently compressed with age is shown by the progressive decrease in width from the first year measurements through the last. This shift is illustrated in the graph by numbering the points to indicate pavement age in years at the time of measurement. The line of regression shows the average relationship between joint width and temperature for the life of the project. However, this progressive decrease in expansion joint width with age did not occur in the stress cured pavement, and the joints oscillated around the initial width depending on whether the temperature was higher or lower than approximately 38 deg. It is believed that this phenomenon was caused primarily by the restriction of joint closure due to early failure of the premolded rubber seal which permitted excessive infiltration of inert material into the expansion joint openings.

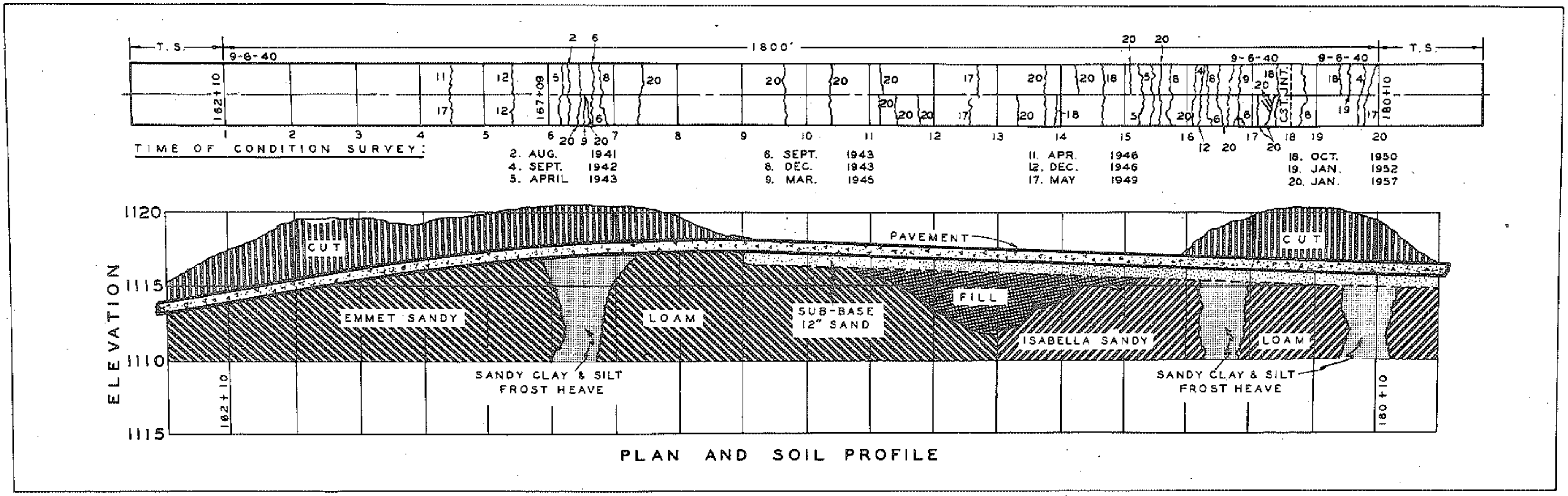
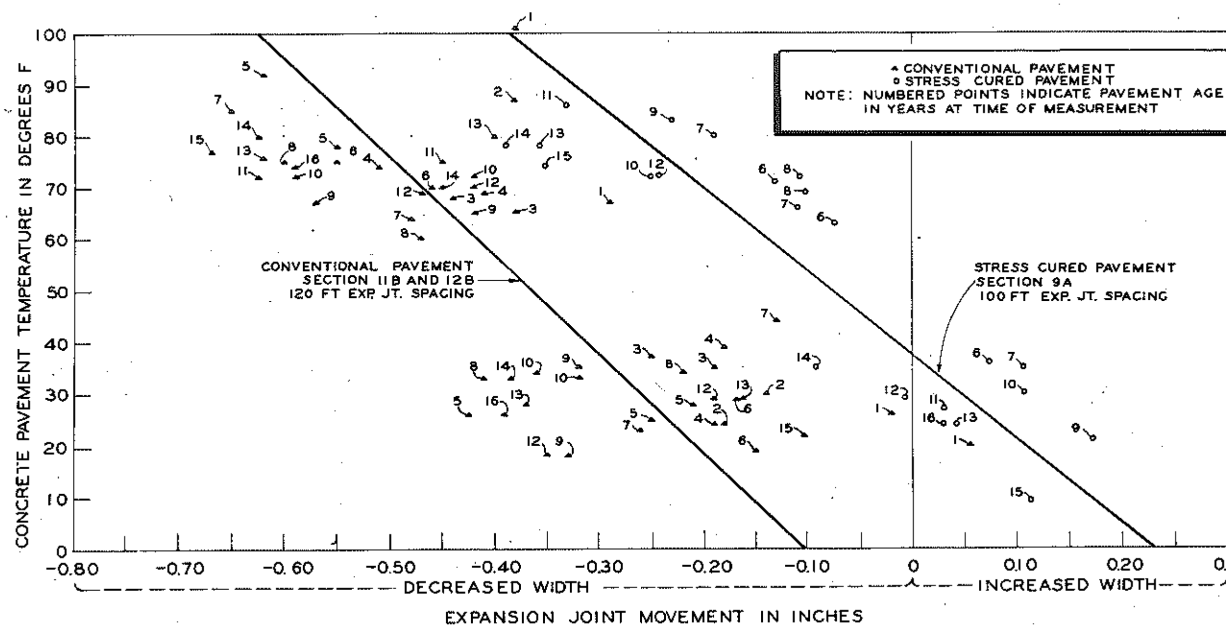


Figure 35.
Stress
Cured
Concrete



EXPANSION JOINT MOVEMENT IN STRESS CURED PAVEMENT COMPARED TO CONVENTIONAL PAVEMENT

CONCLUSIONS

The Design Project is believed to have served its purpose admirably in answering certain questions pertaining to concrete pavement design and construction, which at the time of its inception were of particular interest to the Michigan State Highway Department and other state and federal highway agencies.

Two limitations of this study which may have masked the effects of certain variables on structural performance were the lack of substantial traffic volume and the shortened life due to development of unexpected surface deterioration. The traffic tonnage on the Design Project was only about five percent of that on a more heavily travelled interstate route, US 24 south of Monroe, Mich. Due to pavement scaling which developed rather early in certain areas and eventually required complete resurfacing in 1957, the evaluation terminated after 17 years. The time period, together with the traffic volumes involved, were insufficient to severely test the structural performance of the various design sections.

Conclusions derived from this study which have been reported previously will not be repeated here. The conclusions presented below pertain primarily to the basic objectives of the investigation.

1. Satisfactory performance of long pavement sections of plain concrete pavement with closely spaced contraction joints under full restraint, resulting from the elimination of expansion joints or their spacing at long intervals, indicates that expansion joints are unnecessary except perhaps at such places as intersections, or structures, where excessive compression stresses caused by expansion forces would be undesirable.

2. Elimination of expansion joints in plain concrete pavement construction greatly improves the efficiency of aggregate interlock in preventing joint faulting, but this practice cannot be depended upon to entirely eliminate the need for mechanical load transfer with certain slab lengths, traffic volumes, and subbase conditions.

3. Because of the limitations of the test as stated previously, no general conclusion can be made as to the comparable performance of the reinforced and plain concrete pavement designs included for study in Series 1 through 4--reinforced concrete pavement with different spacings for

warping joints and for contraction joints with dowels, and plain concrete pavement with different spacing for contraction joints with and without dowels. However, four significant facts are evident:

a. Slab length seemed to be the predominant factor in the amount of slab cracking.

b. Joint spacing of approximately 10 ft would be necessary to completely prevent transverse slab cracking. The rate of transverse cracking increased approximately in relation to the square of increased slab length over 10 ft.

c. Plain concrete pavement with dowels at contraction joints performed better than plain concrete pavement without dowels at contraction joints.

d. Even though the longitudinal steel in the reinforced concrete pavement met accepted design criteria, there was evidence of longitudinal steel rupture at warping joints. This breakage developed earlier and to a greater degree in the pavement with 30-ft contraction joints, containing smaller longitudinal wires, than in sections with 60-ft contraction joints, even though the design stress was higher in the latter case. It is quite possible that the principal cause of breakage was corrosion, indicating that the size of longitudinal wire may be more important than design stress in preventing breakage at warping joints.

4. The wide variations in joint width movement found under similar design and climatic circumstances, indicate clearly that average values should be used with caution in determining slab lengths on the basis of expected joint widths. Rather, maximum joint width measurements for specific local conditions should be considered as design criteria in order to ensure satisfactory joint performance.