HIGHWAY RESEARCH BOARD

Research Report 17-B

Joint Spacing in Concrete Pavements:

10-Year Reports on Six Experimental Projects

National Academy of Sciences—
National Research Council

publication 417

The opinions and conclusions expressed in this publication are those of the authors and not necessarily those of the Highway Research Board.

HIGHWAY RESEARCH BOARD

Research Report 17-B

Joint Spacing in Concrete Pavements:

10-Year Reports on Six Experimental Projects

1956

Washington, D. C.

Department of Design

T. E. Shelburne, Chairman
Director of Research, Virginia Department of Highways

COMMITTEE ON RIGID PAVEMENT DESIGN

William Van Breemen, Chairman
Research Engineer, Engineering Research and Soils
New Jersey State Highway Department

A. A. Anderson, Manager, Highways and Municipal Bureau, Portland Cement Association
A. T. Bleck, Assistant State Highway Engineer, State Highway Commission of Wisconsin
Franklin B. Brown, Managing Director, Wire Reinforcement Institute, Inc., National Press Building
W. E. Chastain, Sr., Engineer of Physical Research, Illinois Division of Highways
H. F. Clemmer, Engineer of Materials and Standards, D. C. Engineer Department, District Building
E. A. Finney, Assistant Testing and Research Engineer, Michigan State Highway Department, Room 3, Olds Hall, Michigan State College
A. T. Goldbeck, Engineer Director, National Crushed Stone Association
Robert Horonjeff, Institute of Transportation and Traffic Engineering, University of California,
T. J. Kauer, Chief Engineer, Holmes Construction Co., Wooster, Ohio
O. L. Kipp, Assistant Commissioner and Chief Engineer, Minnesota Department of Highways
L. A. Palmer, Bureau of Yards and Docks Annex, Department of the Navy
G. S. Paxson, Assistant State Highway Engineer, Oregon State Highway Commission, Salem, Oregon
R. W. Peebles, Mississippi State Highway Department,
Thomas B. Pringle, Office, Chief of Engineers, Department of the Army
F. V. Reagel, Engineer of Materials, Missouri State Highway Department
F. H. Scrivner, Spencer J. Buchanan & Associates, Inc., 41 Lancaster Grove, Hampstead, London N.W. 3, England
E. C. Sutherland, Bureau of Public Roads, U. S. Department of Commerce
L. W. Teller, Bureau of Public Roads, U. S. Department of Commerce

Foreword

At the 20th Annual Meeting (1940 of the Highway Research Board a general description was presented of an investigation of "Joint Spacing in Concrete Pavements" that was being instituted as a cooperative research between the respective highway departments of California, Kentucky, Michigan, Minnesota, Missouri and Oregon and the Public Roads Administration. In each State an Experimental Pavement several miles in length, embodying the experimental features has been constructed and kept continuously under observation.

The projects in Kentucky, Michigan and Minnesota are described in Volume 20, Proceedings, Highway Research Board, and those in Oregon and Missouri in Volume 21. The general description of the California project is given with the California report in this bulletin.

In addition to these experimental pavements in service in the several States the program included a study of the structural efficiency of transverse joints of the weakened-plane type to be made by the Public Roads Administration.

Briefly, the experimental features common to the six State projects consist of a series of plain and reinforced concrete sections in which the joint spacing is varied. The plain concrete sections have transverse contraction joints at relatively close spacing (15 to 25 ft.) and expansion joints at 120, 400, 800 and 5280 ft. The reinforced sections have expansion joints at 120 ft. spacing with one intermediate contraction joint.

In general, load transfer devices were used in all expansion joints but were used in only part of the contraction joints of a given project in order to determine whether or not load transfer is needed with closely spaced contraction joints of the weakened-plane type. Several of the States included in their projects additional experimental features of design that were of particular interest to them. These features are described in the reports mentioned above.

Research Report No. 3B "Investigational Concrete Pavements, Progress Reports of Cooperative Research Projects on Joint Spacing" issued in 1945 describes the condition of the pavements at that time.

This report contains progress reports describing the condition of the pavements in the respective states and presents data collected up to the present time. A comparative study of the data is reported by Mr. E.C. Sutherland of the Bureau of Public Roads.

Contents

FOREWORD - iii

ANALYSIS OF DATA FROM STATE REPORTS
Earl C. Sutherland - 1

REPORT ON EXPERIMENTAL PROJECT IN—

CALIFORNIA, F.N. Hvecm - 12

KENTUCKY, D.H. Sawyer - 21

MICHIGAN, H.C. Coons - 35

MINNESOTA, E.C. Carsberg and P.G. Velz - - - - - - - - - - 89

MISSOURI, F.V. Reagel - 142

OREGON, G.S. Paxson - 151

Analysis of Data from State Reports

EARL C. SUTHERLAND, Bureau of Public Roads

●THE several experimental pavements for this investigation were constructed in 1940 and 1941. The pavements in Kentucky, Michigan and Minnesota are described in Proceedings, Highway Research Board, Vol. 20 (1940) and those in Oregon and Missouri in Vol. 21 (1941). The California pavement was described in the 5-year progress report published in 1945 with progress reports of the investigations in the other five states (1). A complete report of the study of the structural efficiency of transverse joints of the weakened-plane type made by the Bureau of Public Roads as a part of this investigation was also included in this same publication (1, 2).

A comparative analysis of the data contained in the six state, 5-year progress reports was published in 1946 (3).

Each of the pavements contained what might be called basic sections which were of essentially the same design in all of the states. Several states also incorporated in their pavements additional designs that are of interest, but do not lend themselves to comparative study. In this summary study only the data obtained from the studies of the basic sections will be included.

The spacing of the expansion and contraction joints in these pavements is shown in Table 1. It will be observed that the majority of the sections are of plain concrete and have contraction joints at intervals of 15 to 25 feet, the spacing in the different pavements being that favored by the respective states. The most important variable in this investigation is the spacing of the expansion joints and as the table shows this varies from an interval of 120 feet to no expansion joints in a mile of pavement.

Further data pertaining to the structural design of these pavements are given in Table 2 and it will be noted that in some details there are a number of differences in the designs of the pavements of the different states. For example, California used a redwood board expansion joint filler while the remainder of the states used preformed bituminous fiber and Missouri used the Translode base load transfer device while the remainder of the states used plain dowels. Also, the amount of expansion space was held constant for the various expansion joint spacings in the pavements of all of the states except Michigan. In this state the amount of expansion space was increased as the distance between the expansion joints was increased. In the more important details the designs of the pavements of the different states are, however, very similar.

In Table 3 are shown the length of the several pavements, the period of construction, the methods used in curing each and the time at which the basic set of measurements were made for determining the joint width changes. It will be noted that (1) with the exception of the California pavement and part of that in Michigan all were laid during the summer months, (2) methods used in curing the pavements varied widely, (3) the time at which the basic joint-width measurements were obtained ranged from immediately after the concrete had taken its initial set to several months after the pavement was laid.

Since the construction of these experimental pavements all of the states have made measurements and observations of the following: (1) daily and seasonal variations in temperature, (2) daily, seasonal and progressive or permanent changes in the widths of the expansion and contraction joints, (3) measurements of faulting at the joints, (4) pumping and (5) the general condition of the pavement.

After the publication of the 5-year reports the schedule of daily and seasonal joint width measurements was greatly reduced but other measurements and observations, including permanent joint width measurements, were continued.

A summary of the traffic data for the several pavements during the first 10 years of their life is shown in Table 4. It will be observed that there has been a moderate amount of heavy truck traffic on the Oregon pavement, but that the amount of truck traffic on the other pavements has been relatively light with the possible exception of the California pavement. The amount of traffic of all types is, however, increasing on all of the pavements.

TABLE 1

SPACING OF EXPANSION AND CONTRACTION JOINTS IN DIFFERENT SECTIONS OF PAVEMENTS

Section No.	Spacing of Joints Expansion	Contraction	Reinforcement
	feet	feet	lb. per 100 sq. ft.
1	1 mile (approx.)	15 to 30 [a]	None
2	800	15 to 30	None
3	400	15 to 30	None
4	120 to 125	15 to 30	None
5	120 to 125	15 to 30	None
6	120	120 [b]	60 or 70
7	120 to 125	15 to 30	None

[a] The spacing was generally the same throughout the length of the respective projects but varied between the different states.
[b] 60-foot joint spacing.

Expansion Joints Closed Progressively

In Figure 1 is shown a comparison of the annual and progressive changes in the widths of the expansion joints in the non-reinforced sections. The annual joint-width changes are indicated by the length of the stippled bars and were computed from data obtained in the winter and summer of each year. Since the data for each annual cycle are plotted with respect to the same basic set of measurements, taken at the time indicated in Table 3, the position of the bars indicates the progressive or permanent changes in widths of the joints. It was not always possible to make measurements at the joints under extreme conditions so that the annual width changes presented are not necessarily the maximum for the yearly cycle. This same limitation also had an influence on the indicated progressive changes in width.

It will be observed that there is a general tendency for the expansion joints to close with time and that the annual change in width of the joints is as a general rule greater during the early life of the pavement than later. The annual changes in width of the joints, shown in this figure, are caused by the combined effect of the annual moisture and temperature changes of the concrete and the progressive change in width of the joint. The change in width caused by the annual moisture and temperature change of the concrete would be expected to be approximately the same each year, but the annual progressive change in width becomes smaller with time. For example, assume that a plain concrete pavement with expansion joints and closely spaced contraction joints is laid during the spring at a reasonably low temperature. As the temperature rises seasonally the pavement will expand, causing the slab units to be shifted over the subgrade toward the expansion joints which results in a progressive closure of those joints. As the temperature of the pavement drops seasonally the slab units will not be shifted

TABLE 2

DESIGN DATA ON THE EXPERIMENTAL PAVEMENT INCLUDED IN THE COMPARATIVE STUDY

State	Cross section	Expansion Joints Width	Filler	Load transfer	Contraction Joints Type	Load transfer	Reinforced section [a] Panel length	Weight of reinforcement
	in.	in.					feet	lb. per 100 sq. ft.
Mich.	9-7-9	1 [b]	Preformed bituminous fiber	Dowels	Flexplane ribbon	Dowels	60 [c]	60
Minn.	9-6-9 7 unif.	1	Preformed bituminous fiber or granulated cork	Dowels except as noted [d]	Grooved, copper water seals and latex-oil mixture in majority	No dowels except as noted [e]	60	
Mo.	9-7-9 9.8-7.8-9.8	1	Preformed bituminous fiber	Translode base	Grooved, pressure injected Tarvia XC	Dowels	60	70
Ky.	9-7-9 7 unif.	1	Dowels	Dowels except as noted [d]	Bituminous fiber strip (sealed)	Dowels	60	70
Calif.	9-7-9 8 unif.	¾	Redwood strips	Dowels	Grooved, poured blended asphalt	Dowels	60	70
Ore.	9-7-9 8 unif.	¾	Preformed bituminous fiber	Dowels	Asphalt impregnated felt strip	Dowels	60	

[a] 120-ft. spacing of expansion joints.
[b] Either 1, 2, or 3 one-inch wide joints, depending on length of subsection.
[c] Divided by dummy joint with reinforcement continuous through joint.
[d] No dowels in uniform-thickness section. This section has expansion joints at 120-ft. intervals.
[e] Dowels in reinforced section and in either one or two of the sections having expansion joints at 120-ft. intervals.

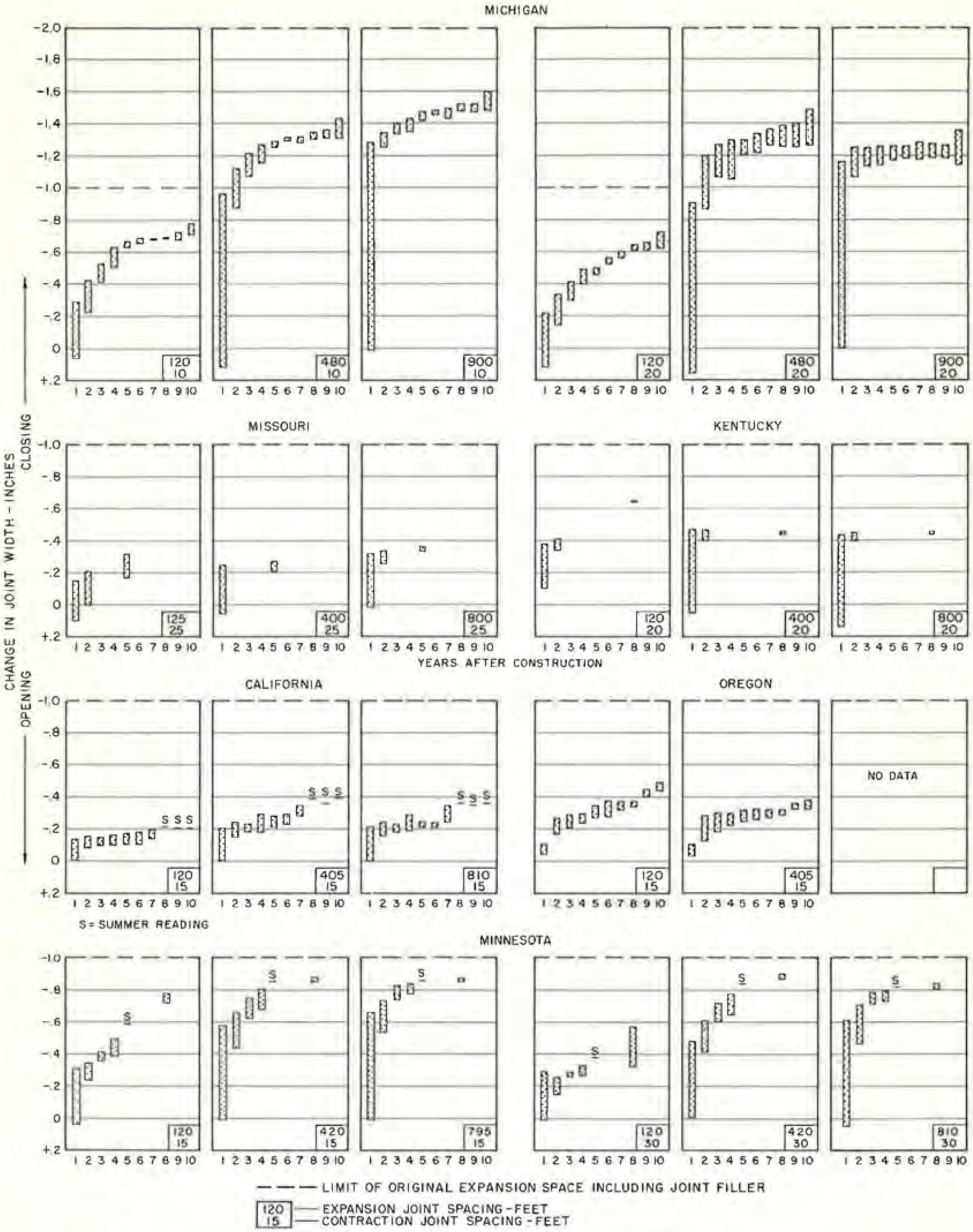


Figure 1. Annual and progressive changes in the width of expansion joints - non-reinforced sections.

over the subgrade, but will expand and contract about their own centers. After the first year there should be little or no progressive closure of the expansion joints resulting purely from temperature changes, but if foreign material infiltrates the contraction joints closure of the expansion joints will continue.

The data from the different states are in general agreement in showing that (1) there is a general tendency for the expansion joints to close with time, (2) the magnitude of

TABLE 3

CONSTRUCTION DATA ON THE EXPERIMENTAL PAVEMENTS

State	Length mi.	Time of laying concrete Year	Month	Method of curing	Time of basic measurements at joints
Michigan	10.7	1940	July 31 to Oct. 25	Burlap and straw for a total of 7 days	Immediately after completion of each section
Minnesota	8.1	1940	Aug. 6 to Sept. 20	Impermeable fiber filled paper for 72 hr.	Early in Oct. 1940
Missouri	7.0	1941	June to early Aug.	Transparent membrane sprayed on pavement	August 1941
Kentucky	6.3	1940	July 8 to Aug. 16	Burlap and Sisalkraft paper for a total of 4 days	Nov. 27, 1940
California	5.7	1941	Sept. 20 to Oct. 29	Moist earth for 8 days	Feb. 1942
Oregon	3.8	1941	June 10 to July 7	Wet cotton mats for 72 hr.	Immediately after concrete had taken initial set

the permanent closure of expansion joints appears to increase with an increase in the spacing, but the influence of spacing is small for intervals greater than approximately 400 feet and (3) the amount of closure increases with the amount of expansion space.

The magnitude of the progressive expansion joint closure varies considerably between the pavements of the different states. These differences are probably caused by a combination of several factors. For example, the amount of expansion joint closure would be expected to be greater in a pavement which took its final set at a relatively low temperature than in one which took its final set at a high temperature. Also, the amount of expansion joint closure is influenced by the resistance offered to closure by the expansion joint. In this connection the redwood expansion joint filler used in the California pavement offered more resistance to closure than the plastic fillers used in the other states. Also, it is probable that the Translode load transfer devices used in the Missouri pavement offered more resistance to closure than the plain dowels used in the other states.

Other factors which may have had some influence on the amount of expansion joint closure which developed at the expansion joints of the pavements of the different states are (1) amount of available expansion space, (2) climatic differences and (3) differences in the amount of infiltration and, therefore, opening which developed in the intermediate contraction joints.

Contraction Joints Opened Progressively

The annual and progressive changes in widths that occurred in the contraction joints of the nonreinforced sections are shown comparatively in Figure 2. These width changes are with respect to the basic set of measurements made at the time indicated in Table 3 and are averages for a number of joints in the central parts of the sections some distance from the expansion joints. The differences in the positions of the annual joint-width change bars with respect to their base line, in the different states, is explained by the fact that the basic measurements were made at different temperature conditions and at different times with respect to cracking of the weakened-plane joints.

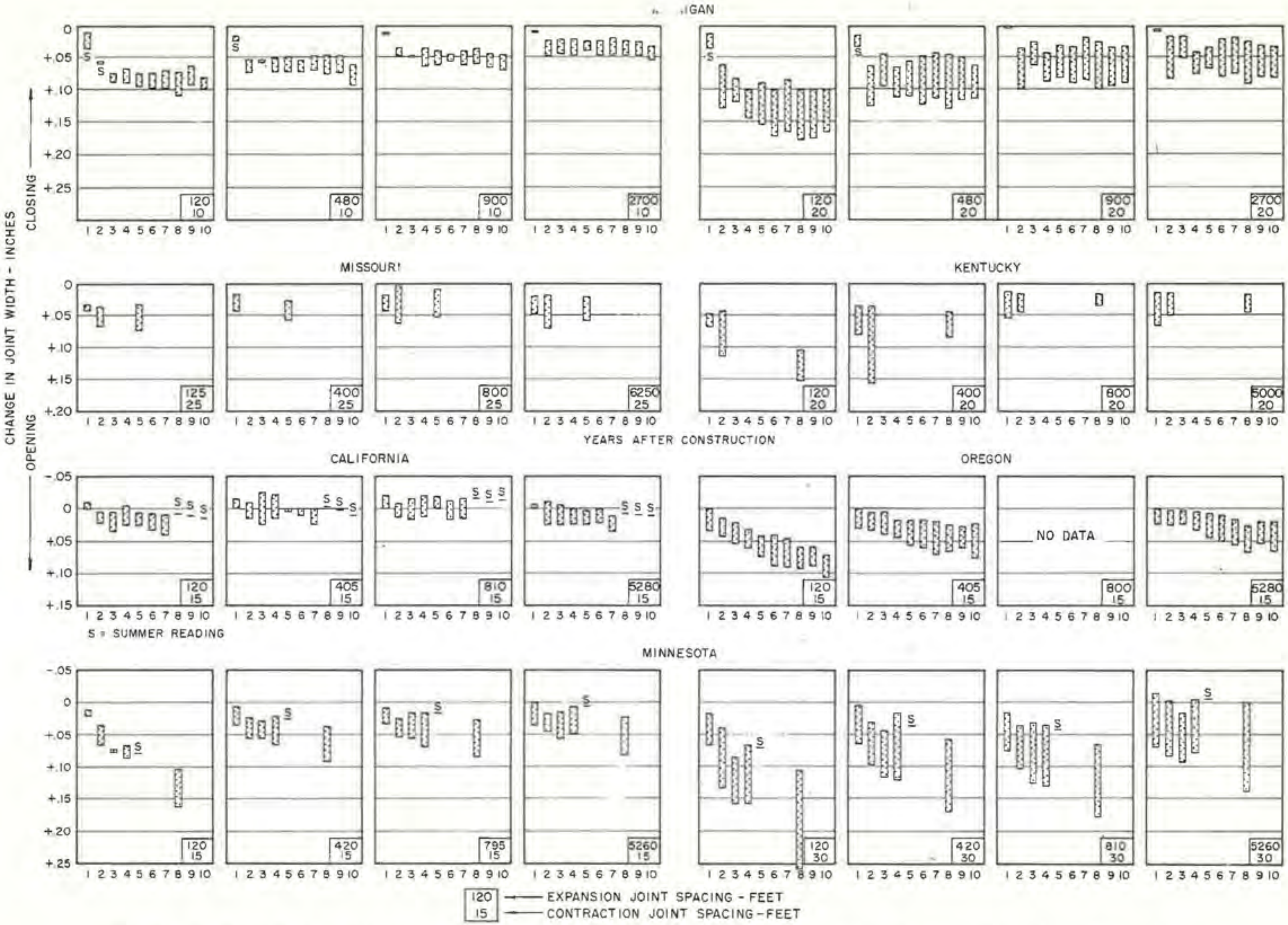


Figure 2. Annual and progressive changes in the width of contraction joints - non-reinforced sections.

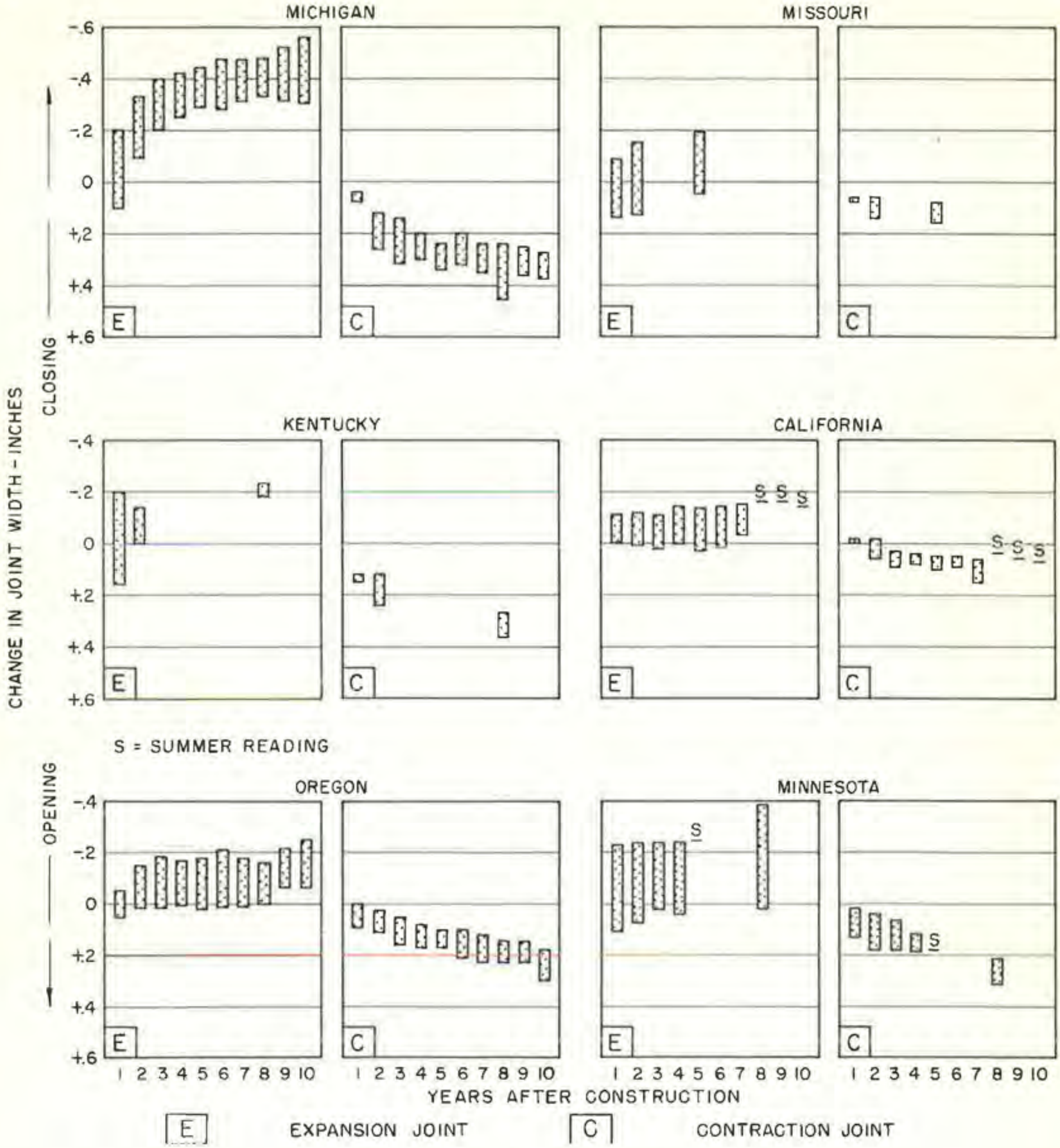


Figure 3. Annual and progressive changes in the width of expansion and contraction joints - reinforced sections.

As indicated in Table 3 the basic measurements, or those used for computing the subsequent changes in joint width, were made at the upper range in seasonal temperatures in Missouri and Oregon, at an intermediate range in Michigan and Minnesota and at a lower range in Kentucky and California.

Also the basic measurements were made immediately after the concrete had taken its initial set in the States of Michigan and Oregon and later after an undetermined number of joints had fractured in the remaining states. Thus, in the States of Michigan and Oregon the joint width changes shown are the total changes, but this is not necessarily true for the other states. At the end of the first year, the magnitude of the joint movements as related to the basic measurements indicated that practically all, if not all, of the joints had fractured.

There is a close relationship between the progressive closures which develop at the expansion joints of concrete pavements and the progressive openings which develop at the contraction joints. Actually the progressive or permanent closure of the expansion joints is essentially the accumulative progressive openings which developed at the intermediate contraction joints. Because of this interrelationship many of the factors mentioned earlier as influencing the progressive closing of the expansion joints apply equally to the progressive opening of the contraction joints.

It will be observed in Figure 2 that (1) the contraction joints opened progressively with time, the rate being greatest during the early life of the pavement, (2) the amount of progressive opening was greatest in the sections with the 120-foot expansion joint spacing, but did not increase appreciably with expansion joint spacing where the spacing was approximately 400 feet or greater and (3) the annual joint width change of the contraction joints increased with an increase in spacing of the contraction joints. (See Michigan and Minnesota data.)

The fact that the progressive changes in widths of the contraction joints varies between the different states is, as explained earlier, associated with the same factors that influenced the progressive movements at the expansion joints. These are (1) the temperature of the concrete in the different pavements at the time of initial set, (2) temperature conditions at the time that the basic measurements were made, (3) differences in the resistance offered to closure of the expansion joints by the load transfer devices and joint fillers, (4) amount of available expansion space and (5) amount of infiltration of foreign material which developed in the joints.

Movements at the Expansion and Contraction Joints of the Reinforced Sections

The annual and progressive changes in the widths of the expansion and contraction joints in the reinforced sections of the different pavements are shown in Figure 3. Excepting the Michigan pavement these sections are divided into 60-foot panels by alternate expansion and contraction joints, the steel being interrupted at the joints. The Michigan pavement differs from the others in that the 60-foot panels are divided at the center with a warping joint through which the steel is continous.

As in the plain concrete sections a progressive closing of the expansion joints and a progressive opening of the contraction joints has developed. Generally the annual changes in width of the expansion joints are greater than those of the contraction joints. Since the same type of load transfer devices were used in both expansion and contraction joints in all cases except the Missouri pavement, this behavior can hardly be attributed to differences in resistance offered by these devices. This same phenomenon was observed and studied in the Arlington investigation. It appears to be caused by a shifting of the slabs over the subgrade as the temperature of the slabs changes, causing a greater concentration of movement at the expansion than at the contraction joints. This is discussed at greater length in the report on the Arlington investigation (4).

The progressive and annual changes in widths of the joints vary among the different pavements and, except for magnitude, the variations are similar to those found in the plain concrete sections. The probable reasons for these variations have been mentioned in the discussion of the joint width changes for the plain concrete pavements.

A comparison of the maximum openings observed at the contraction joints separating the 60-foot panels of the 120-foot reinforced sections with maximum openings observed at the contraction joints separating the shorter panels of the 120-foot plain concrete sections is shown below:

State	Ratio of slab lengths [a]	Ratio of maximum contraction joint opening
Michigan	3: 1	2. 5: 1
Missouri	2. 4: 1	2. 3: 1
Kentucky	3: 1	2. 5: 1
California	4: 1	3. 8: 1
Oregon	4: 1	3: 1
Minnesota	4: 1	1. 9: 1

It is evident that while the openings of the contraction joints in the reinforced sections are larger than those in the plain concrete sections the differences are not, in all cases, proportional to the slab lengths.

Most of the daily joint width measurements were made during the first five years

[a] Ratio of slab lengths in reinforced sections (60 ft.) to slab lengths in nonreinforced sections.

TABLE 4

SUMMARY OF TRAFFIC DATA FOR THE SEVERAL PAVEMENTS

	Average daily traffic									
	1941	1942	1943	1944	1945	1946	1947	1948	1949	1950
					Michigan					
Total traffic	1,590	829	578	733	803	1,204	1,176	1,361	1,472	
Light trucks	43	44	14	32	16	4	5	29	6	
Medium trucks	26	37	56	51	27	30	28	25	32	
Heavy trucks	43	12	13	5	5	7	18	5	5	
Trailer combinations	41	68	101	62	89	46	59	64	61	
					Missouri					
Total traffic	761	605	544			943	1,052	1,052		1,330
Light trucks	68	54	48			84	93	93		118
Single unit trucks	145	115	103			180	200	200		254
Trailer combinations	19	15	14			24	26	26		33
Buses	8	6	5			9	11	11		13
					Kentucky					
Total traffic	840	649	648	700	750	1,003	1,068	1,140	1,066	1,400
Light trucks	237	413	300	325	333	363	300	282	194	255
Medium trucks	0	4	64	63	29	7	61	149	175	230
Heavy trucks and trailer combinations	8	8	11	12	16	28	22	18	2	3
Buses	11	17	9	10	12	15	15	16	14	18
					California [a]					
Total traffic	1,850	1,550	1,420	1,670	1,720	2,300	2,450	2,770	3,120	3,240
Commercial vehicles	500	510	520	640	420	560	660	730	760	830
					Oregon					
Total traffic	3,810	4,170	4,200	3,865	4,440	5,210	5,770	6,345	7,150	7,300
Light trucks	169	184	205	212	257	276	294	324	361	406
Heavy trucks	220	262	277	290	341	411	368	339	379	439
					Minnesota					
Total traffic	572			602		682		1,274		1,580
Single unit trucks	84			160		126		144		251
Heavy trucks and trailer combinations	2			11		38		45		109
Buses	2			2		3		4		5

[a] For this state the number of vehicles indicated is for 16 hours.

and are published in the 5-year reports. For a discussion of the daily joint width changes the reader is referred to the 5-year progress reports and the comparative study of these reports referred to earlier.

Only a Moderate Amount of Faulting Has Developed

As indicated earlier in the discussion of Table 4 none of these pavements has carried more than a small to moderate amount of heavy truck traffic. Thus it would not be expected that serious pumping and faulting would develop. Furthermore, the pavements of California, Michigan and Oregon were laid either on granular subgrades or subbases which would be expected to control pumping.

The faulting data reported by the states are summarized for the expansion and contraction joints in Tables 5 and 6, respectively. It is apparent from Table 5 that mechanical load transfer devices have been very helpful in controlling faulting at expansion joints even on pavements with granular subbases.

The data pertaining to faulting at the contraction joints, Table 6, are inconclusive, due apparently to the small amount of faulting which has developed thus far in the contraction joints. The States of Missouri and Minnesota, however, made special studies, the data from which are not included in the above tabulation, but which show that mechanical load transfer is helpful in reducing faulting in contraction joints of pavements with closely spaced contraction joints and with expansion joints at intervals of approximately 120 feet.

With regard to pumping, three of the states report that none was observed while the remaining three do not mention it. Apparently no significant pumping has developed in any of the pavements.

The only reason for placing expansion joints in concrete pavements is to prevent structural damage caused by high compressive forces. In each of the six experimental pavements, under discussion, there is a one-mile section with contraction joints at intervals of 15 to 25 feet and no expansion joints. Also there are two sections in each pavement with contraction joints at the same intervals and expansion at intervals of approximately 800 feet.

Five of the states report no blow-ups in the ten-year reports while one, Minnesota, reports one. Thus it is strongly indicated that expansion joints, except in special locations, can be eliminated in plain concrete pavements of sound concrete which does not develop excessive permanent growth and with contraction joints at intervals of 15 to 25 feet. In this connection four of the states drew conclusions which, in effect, state that where the aggregates are of sound character expansion joints are not required in concrete pavements except at bridge approaches, intersections, etc. The other two states drew no conclusions, feeling that on the basis of developments in their pavements up to this time conclusions were not justified.

Only a nominal amount of cracking has developed in these pavements up to this time. The amount of transverse cracking appears to bear little or no relationship to the spacing of the expansion joints, but is of course directly related to the panel length. It is indicated that a panel length exceeding approximately 20 feet is too great to control cracking in plain concrete pavements.

All of the states express the opinion that the pavements should be observed for a greater length of time before drawing any conclusions concerning the relative merits of the plain and the reinforced sections.

TABLE 5

FAULTING AT EXPANSION JOINTS WHEN PAVEMENTS WERE APPROXIMATELY 10 YEARS OF AGE

Section No.	Spacing of joints		Type of load	Joints faulted		
	Expansion	Contraction	transfer	⅛"	⅛" to ¼"	Over ¼"
	feet	feet		%	%	%
			Michigan			
10 A-1	120	20	Dowels	5	0	0
10 A-2	120	15	Dowels	0	0	0
10 B-1	120	20	None	33	39	0
10 B-2	120	15	None	28	34	11
			Missouri			
5-a 5 R	125	25	Translode	95	5	0
6-a 6 R	120	60	Translode	94	6	0
7-a 7 R	125	25	None	76	17	7
			Oregon		inches	
5	120	15	Dowels		.048[a]	
6	120	60	Dowels		.048	
7	120	15	None		.084	

[a] Faulting values are averages for expansion joints.

TABLE 6

FAULTING AT CONTRACTION JOINTS WHEN PAVEMENTS WERE APPROXIMATELY 10 YEARS OF AGE

Section No.	Spacing of joints		Type of load	Joints faulted		
	Expansion	Contraction	transfer	$\frac{1}{8}$"	$\frac{1}{8}$" to $\frac{1}{4}$"	Over $\frac{1}{4}$"
	feet	feet		%	%	%
			Missouri			
1	None	25	None	93	6	-
2	800	25	None	91	8	-
3	400	25	None	91	8	-
5	125	25	Dowels	100	0	0
6	120	60	Dowels	100	0	0
7	125	25	None	86	10	4
			Oregon		inches	
1	None	15	None		.060[a]	
3	405	15	None		.060	
4	120	15	None		.060	
5	120	15	Dowels		.048	
6	120	60	Dowels		.060	
7	120	15	None		.072	
			Kentucky		percent	
1	None	20	None	-	10	-
2	800	20	None	-	20	-
3	400	20	None	-	17	-
4	120	20	None	-	10	-
5	120	20	Dowels	-	16	-
6	120	60	Dowels	-	27	-
7	120	20	None	-	26	-
			Minnesota		percent[b]	
1	5,280	20	None		4	
2	800	20	None		4	
3	400	20	None		4	
-	120	20	None		4	

[a] Faulting values are averages for contraction joints.
[b] Percent of joints faulted without reference to magnitude of faulting.

SUMMARY

This investigation was initiated in 1940 as a cooperative effort by six states and the Bureau of Public Roads to obtain information as to the need for expansion joints in concrete pavements. The experimental pavements constructed for this investigation were widely dispersed and covered a wide range in subgrade as well as climatic conditions.

It was found that in pavements with expansion joints spaced at what was considered to be a desirable interval and intermediate contraction joints at sufficiently close intervals to control transverse cracking there was a tendency for the expansion joints to close progressively and the contraction joints to open progressively with time. These movements progress rapidly during the early life of the pavement and within a few years are of sufficient magnitude to destroy aggregate interlock in contraction joints of the weakened plane type. Where the expansion joints were eliminated or widely spaced there has been little or no tendency for the contraction joints to open progressively.

On the basis of the 5-year progress reports the practice of many of the states with respect to expansion joints has changed. Today practically every state has eliminated expansion joints in nonreinforced concrete pavements except at structures and other special locations. This has resulted in pavements which offer greater resistance to pumping and faulting because of the better maintenance of aggregate interlock in the contraction joints.

References

1. Highway Research Board, Research Report No. 3B (1945).
2. Public Roads, April-May-June 1945.
3. "A Comparative Study of Data from the Cooperative Investigation of Joint Spacing." Proceedings, Highway Research Board, Vol. 26 (1946).
4. Public Roads, Vol. 16, No. 9, November 1935.

Report on Experimental Project in California

F. N. HVEEM, Materials and Research Engineer,
California Division of Highways

● The California project to investigate the effect of joint spacing in concrete pavement follows the scope outlined by E. F. Kelley, Chief of the Division of Physical Research, Public Roads Administration. (1) A previous report was made on this project in 1945 by T. E. Stanton, Materials and Research Engineer, California Division of Highways. (2) The following report brings up to date the traffic count, joint movement, rainfall and other data shown in the tables.

Location and General Description

This project is located in the Santa Clara River Valley, Ventura County, about 20 miles from the Pacific Ocean. The pavement was constructed in October 1941, and consists of seven different arrangements of the joint intervals. All experimental sections were constructed in duplicate. The entire 5.65 miles of pavement under the contract was designed as a test section.

The primary purpose of this experiment was to study a series of various expansion and contraction joint spacings.

Details of construction and installation of all of the various test devices are to be found in Mr. Stanton's report.

Tables and Figures

The following tables and figures carry the same titles and numbers as those in the 1945 report. Table No. 10, "Average Change in Joint Widths, Permanent" and Figure No. 18, "Traffic Trend" have been added.

Attention is called to the fact that from 1948, only the August joint readings were made.

Joint Movement

Early in the life of the project, it was observed that certain joints gave somewhat erratic readings for no obvious reason. It was also noted that contraction joints nearest to an expansion joint displayed the least tendency to follow the characteristic daily or seasonal pattern. Therefore, it was decided early to list the movements of expansion joints as one group. The joints next to the expansion joints are listed separately in Table No. 10 under the heading "Adjacent Joints" while the third group covers all the intermediate joints not included in the first two categories. As was to be expected, the greatest movement occurred at the expansion joints. Those sections with 120 foot expansion joint spacing averaged about 0.10 inch less than the longer spacings.

It may also be noted that during the entire ten year period the only evidence of progressive movement or displacement has occurred at the expansion joints which is not surprising in view of the fact that there is no mechanism for forcing the pavement back to its original position.

Seasonal joint movement in both the adjacent joints and the intermediate group was limited to an average of about 0.05 inches. The magnitude of daily joint movement depended upon the daily temperature range at the time the readings were taken.

General Condition Survey

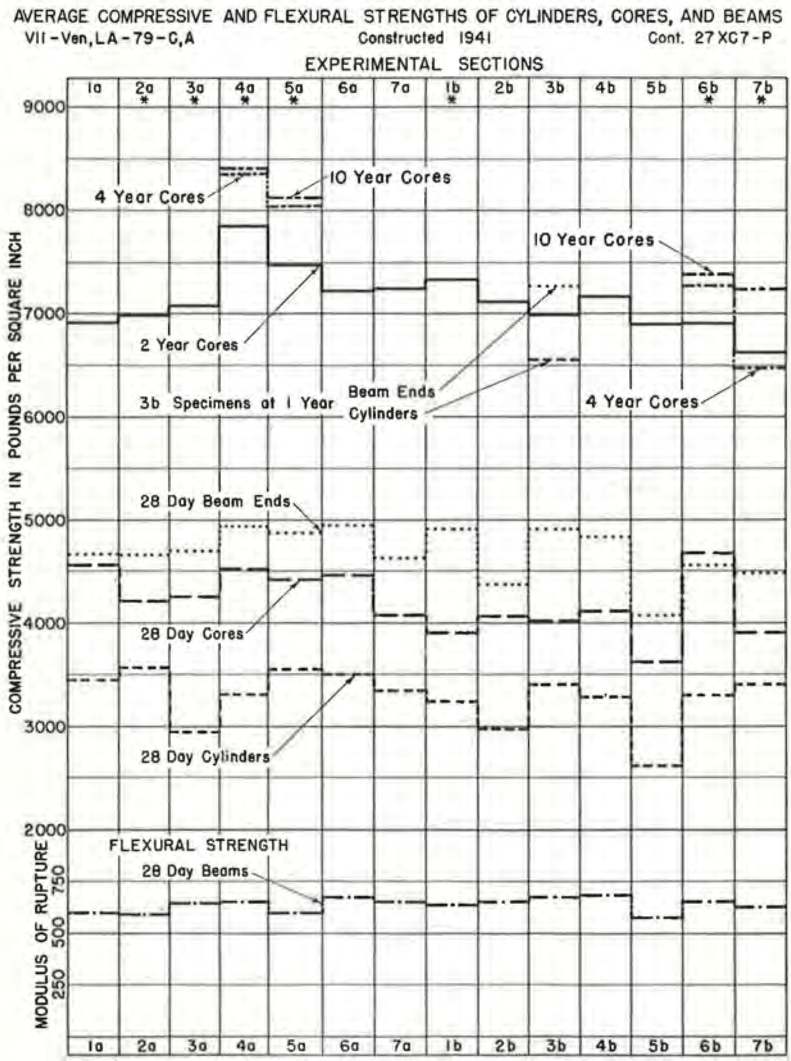
As was reported in 1945, there is little evidence of any type of surface failure. There is no evidence of crushing or spalling due to the daily movement of the concrete slabs. There is no evidence of "Step-off" or of "Pumping" at any of the joints. The riding quality of the entire job is excellent.

The general consition survey is, at this time, essentially the same as in 1945.

Avg. Compressive Strength, PSI

	Cylinders	Cores
4 days	1,210	
10 "	2,050	
28 "	3,610	4,240
3 months	5,210	
6 "	6,010	
1 year	6,570	
2 years		7,135
4 y "		7,555
10 "		7,690

NOTE: Investigational Concrete Pavement in California by Thomas E. Stanton Materials and Research Engineer, California Division of Highways.

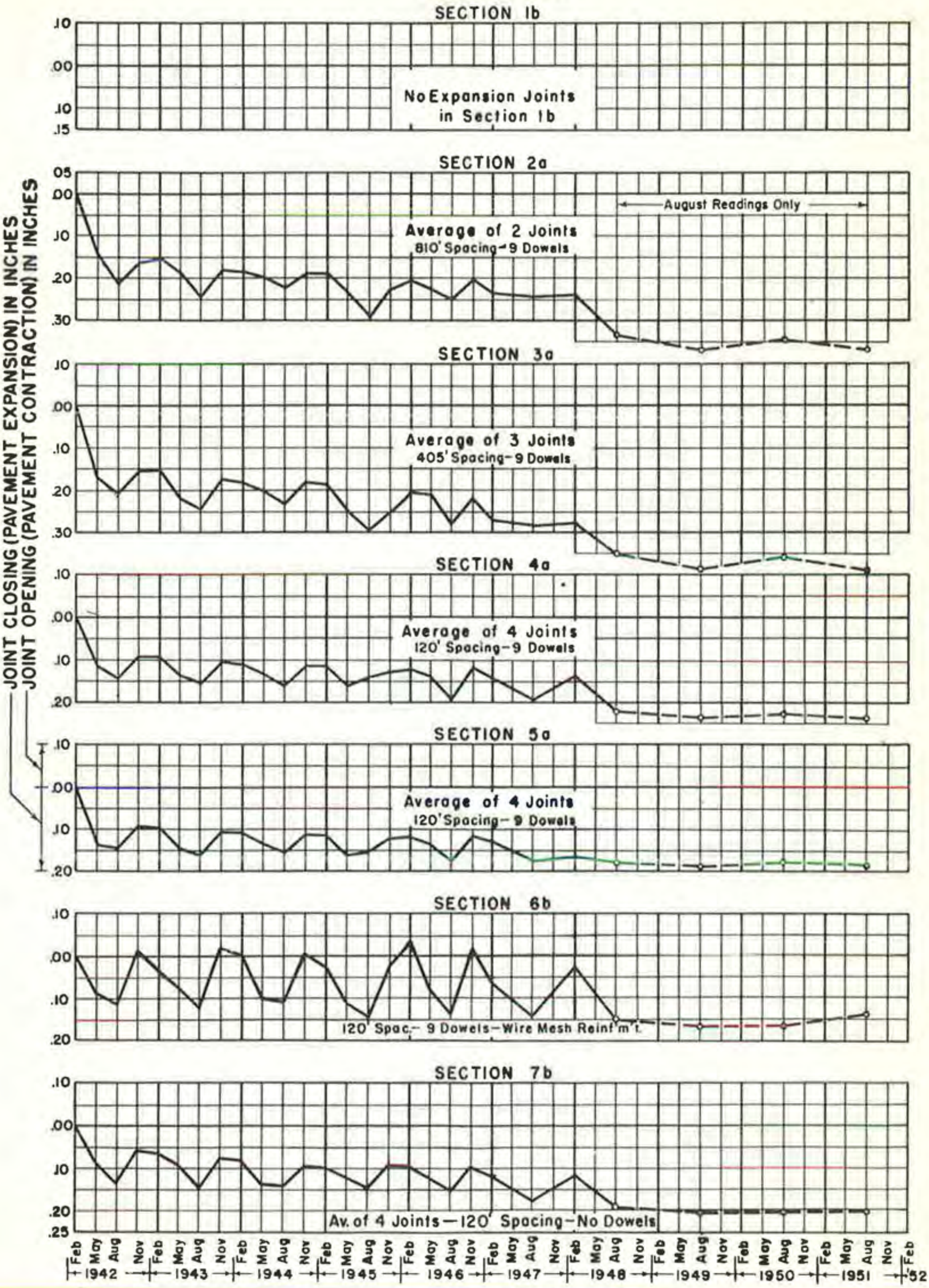


* Gauge and Level Points placed in these sections.

Figure 4.

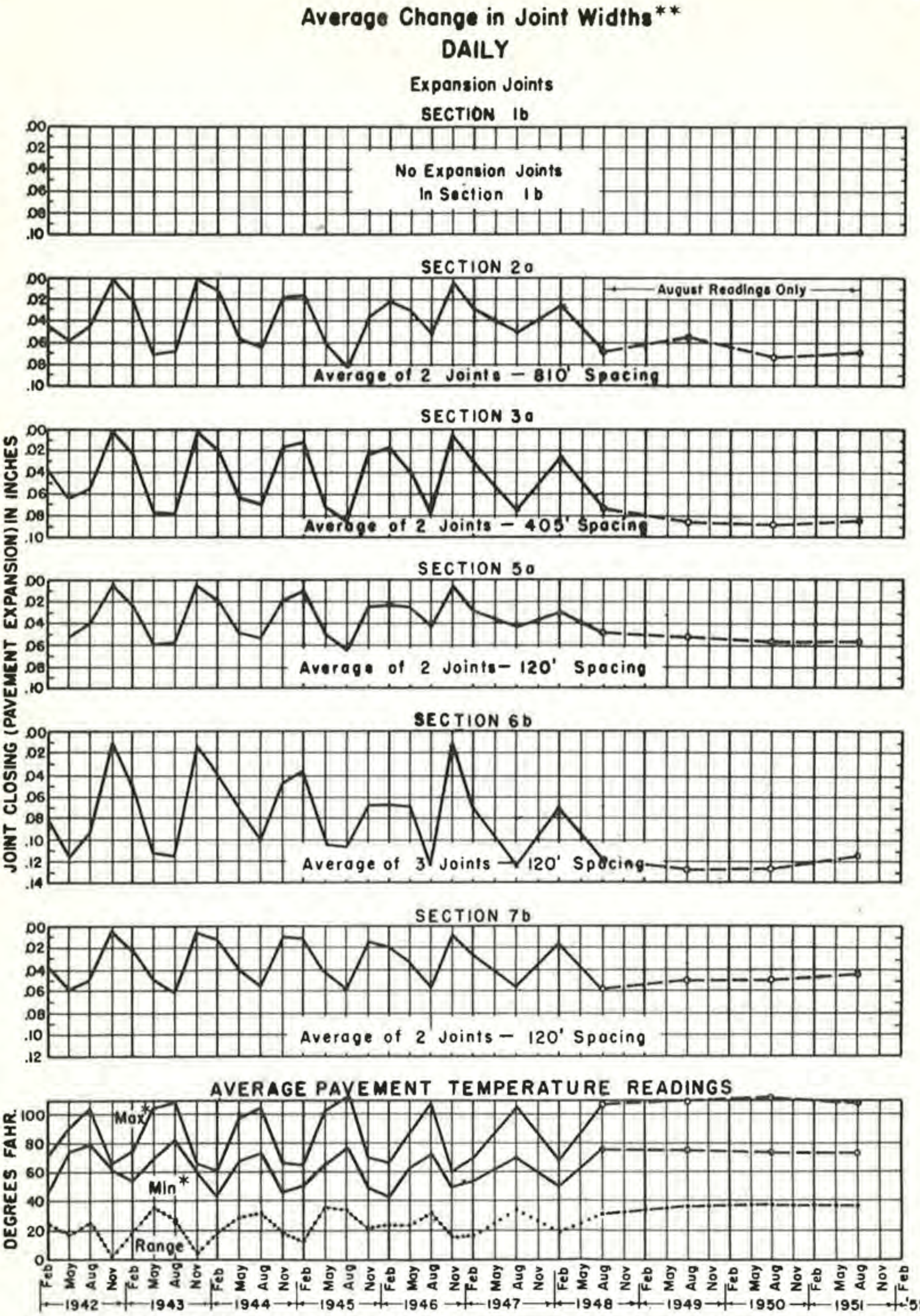
Average Change in Joint Widths *
SEASONAL

Expansion Joints



* NOTE: Co-operative Research Project on Joint Spacing—Venture County, California.

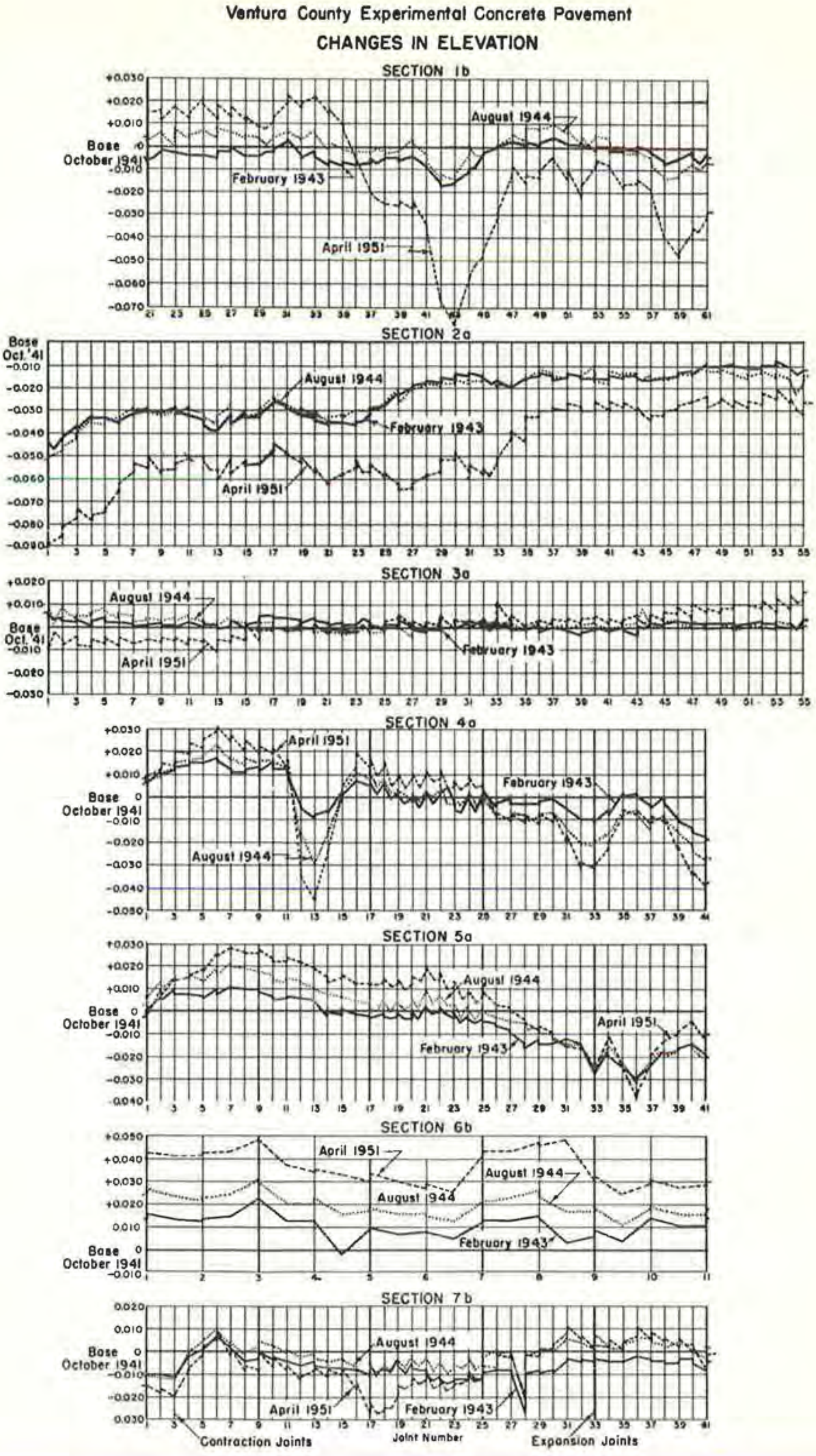
Figure 7.



* Maximum and minimum temperature recordings are for the day measurements were made and are not the maximum and minimum of the month.

** Co-operative Research Project on Joint Spacing- Ventura County, California.

Figure 8.



Ventura County Experimental Concrete Pavement
CHANGES IN ELEVATION
SECTION 1b
August 1944
February 1943
April 1951
Base October 1941
SECTION 2a
Base Oct. '41
August 1944
February 1943
April 1951
SECTION 3a
August 1944
April 1951
February 1943
SECTION 4a
April 1951
February 1943
August 1944
SECTION 5a
August 1944
April 1951
February 1943
SECTION 6b
April 1951
August 1944
February 1943
SECTION 7b
August 1944
April 1951
February 1943
Contraction Joints
Joint Number
Expansion Joints

Figure 9.

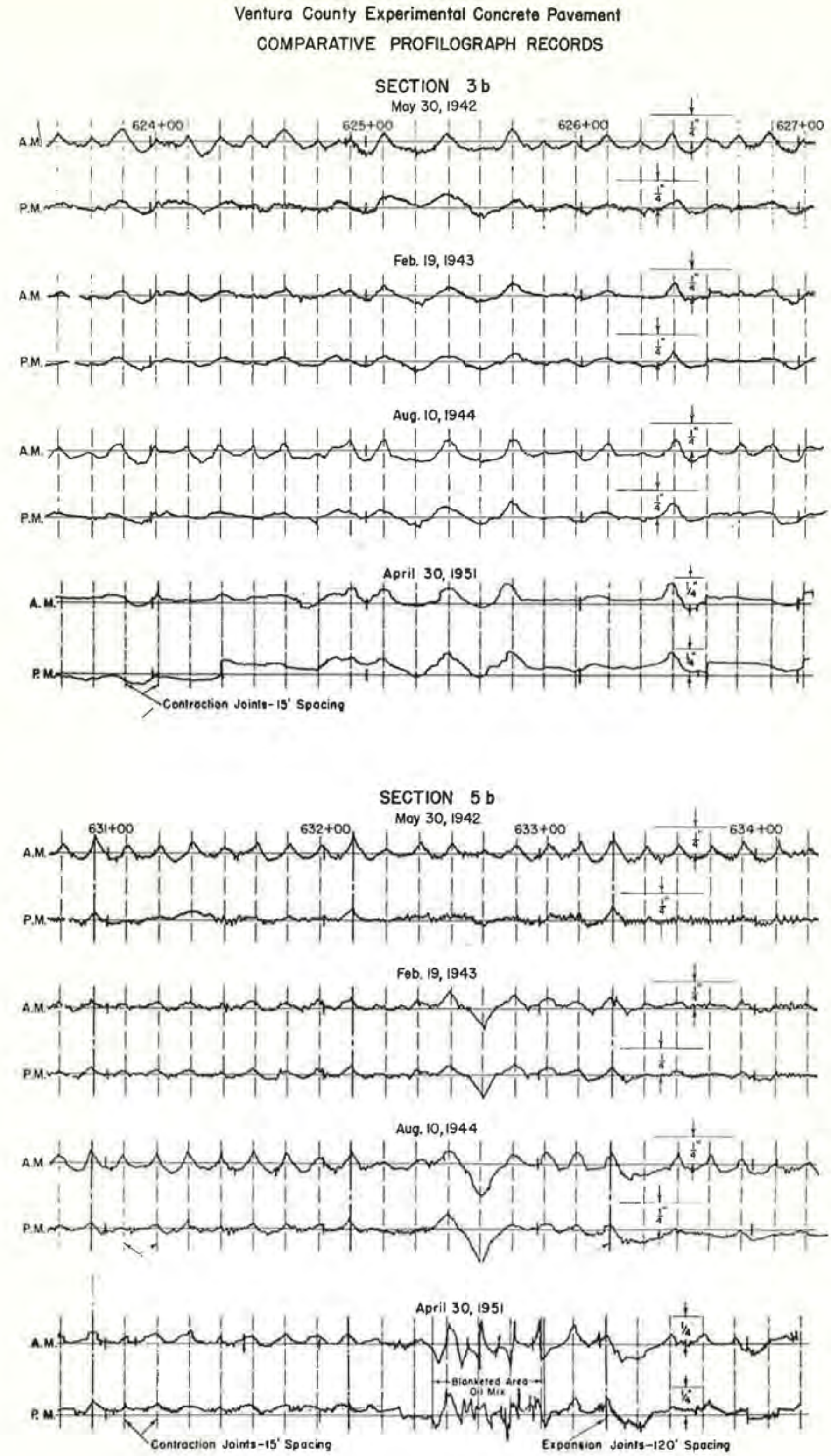

Ventura County Experimental Concrete Pavement
COMPARATIVE PROFILOGRAPH RECORDS
SECTION 3 b
May 30, 1942
Feb. 19, 1943
Aug. 10, 1944
April 30, 1951
624+00
625+00
626+00
627+00
A.M.
P.M.
Contraction Joints-15' Spacing
SECTION 5 b
631+00
632+00
633+00
634+00
Blanketed Area Oil Mix
Contraction Joints-15' Spacing
Expansion Joints-120' Spacing

Figure 16.

Ventura County Experimental Concrete Pavement

COMPARATIVE PROFILOGRAPH RECORDS

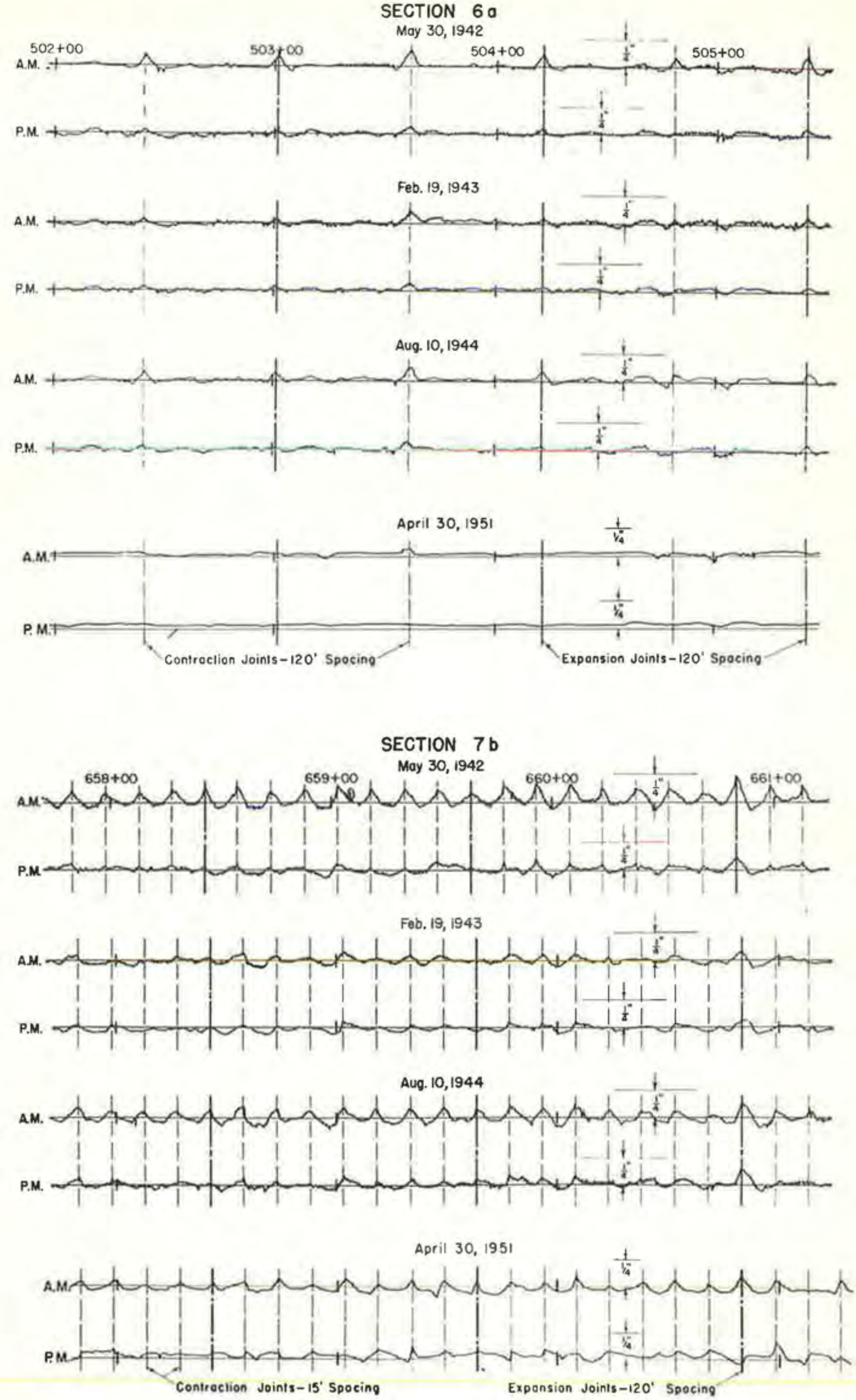


Figure 17.

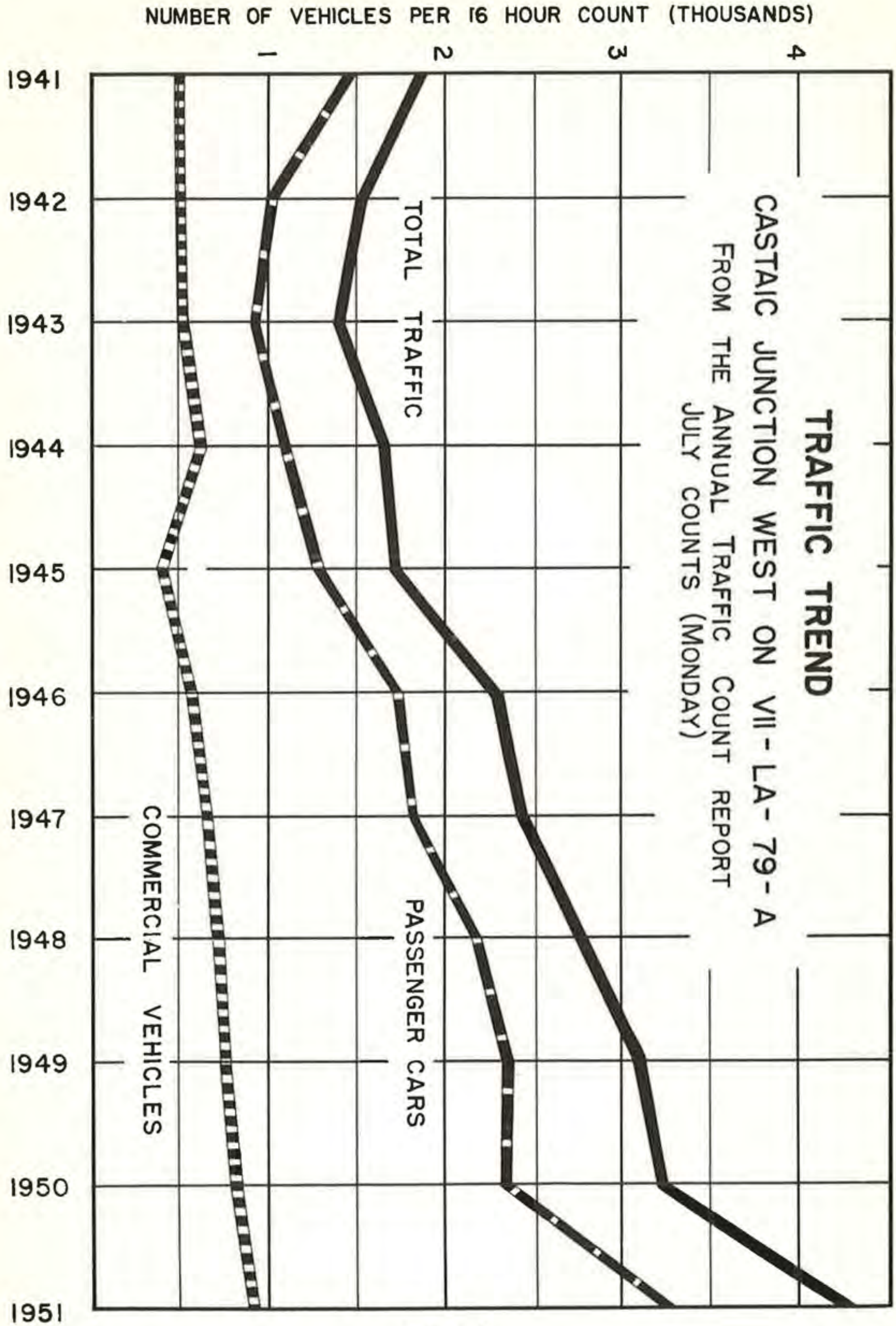

NUMBER OF VEHICLES PER 16 HOUR COUNT (THOUSANDS)
1
2
3
4
1941
1942
1943
1944
1945
1946
1947
1948
1949
1950
1951
TRAFFIC TREND
CASTAIC JUNCTION WEST ON VII - LA - 79 - A
FROM THE ANNUAL TRAFFIC COUNT REPORT
JULY COUNTS (MONDAY)
TOTAL TRAFFIC
PASSENGER CARS
COMMERCIAL VEHICLES

Figure 18.

References

1. "History and Scope of Cooperative Studies of Joint Spacing in Concrete Pavements," Proceedings, Highway Research Board Vol. 20, page 333 (1940).

2. "Investigational Concrete Pavement in California," Highway Research Board Research Reports No. 3B, page 1 (1945).

Report on Experimental Project in Kentucky

D. H. SAWYER, Research Engineer
Kentucky Department of Highways

● THE Kentucky Test Road was constructed during the summer of 1940, by the Kentucky Department of Highways in cooperation with the Bureau of Public Roads. The investigational pavement, a part of US 231 (formerly Ky. 71) is 6. 27 miles in length and is located approximately six miles southeast of Owensboro, in Daviess County.

This report presents a discussion of pavement performance and observations over a 10-year period, beginning in September, 1940. A complete discussion of the original scope, purpose, and early performance of this cooperative project has been given in previous reports (1, 2, 3).

GENERAL FEATURES

The general arrangement and design features of the pavement are given in Table 1. Essentially, there were seven sections with experimental variables which conformed to the general test program, and an added section designated as Standard. This section represented the design used by Kentucky at that time, and for the most part it was constructed over swampy land considered unsuitable for an experimental pavement.

Expansion joints were constructed to accommodate a 1-inch width of premolded bituminous fiber filler and contraction joints were of the weakened plane type with a premolded bituminous fiber filler.

Dowels for load transfer were ¾-inch plain round bars and secured in proper spacing and alignment by welded dowel spacers which remained in place. Spacing of expansion joints in the different sections varied from 60 feet to 5,040 feet in length and contraction joint spacing was 20 feet in all but two sections. In Section 6, the joint interval was 60 feet, alternating with contraction and expansion joints. In the Standard Section these joints were spaced at 30 foot intervals. Sections where wire mesh reinforcing was installed had the initial pour of concrete struck off 2 inches below grade for placing the mesh.

Soil Conditions

Soils throughout the project were predominantly H. R. B. A-4 or approximately A-4-6 materials. Generally speaking, they were uniformly of a fine sand or silty texture, with the clay content (-. 005mm) in all but a few cases lower than 20 percent. These characteristics reflected the derivation of the soil, which was associated with wind transportation.

Soils in the moderate upland usually consisted of windblown fine sands and silts, and only in a few spots did the underlying shale formation have an influence at subgrade elevation. In close association, but pertinent to a relatively small portion of the road, were the silty soils of the lowlands which originated through deposition in a lake created by glacial activity to the north about the same time that comparable windblown soils were deposited in the upland. So far as tests results were concerned, the soils from all parts of the road were quite similar. Fills through the lowlands kept the grade high enough to make internal drainage conditions relatively similar also.

Physical Properties of Concrete

A single brand of Type I Portland Cement was used throughout the project. Fine and coarse aggregates selected for use were Ohio River sand and gravel dredged from a well known bar approximately 8 miles upstream from Owensboro.

The average compressive strength for 68 specimens at 28 days of age, representing one cylinder for each 500 linear feet of pavement, was 4,910 psi. Maximum and minimum strength were 6,200 and 3,890 psi. respectively, and 71 percent of the strengths were within 10 percent of the average strength. The average modulus of rupture was 1,000psi. at 28 days. Maximum and minimum values were 1,200 and 815 psi. respec-

TABLE 1

DESIGN OF EXPERIMENTAL JOINT SECTIONS

Section No.	Length	Design Section	Wire Mesh Reinf.	Expansion Joints		Contraction Joints	
				Spacing	Load Transfer	Spacing	Load Transfer
	ft.	in.		ft.		ft.	
7	1250	7-7-7	None	120	None	20	None
6	1500	9-7-9	70 lb.	60 alt.	Dowels	60 alt.	Dowels
5	1500	9-7-9	Nonc	120	Dowels	20	Dowels
4	1500	9-7-9	None	120	Dowels	20	None
3	2500	9-7-9	None	400	Dowels	20	None
2	3000	9-7-9	None	800	Dowels	20	None
1	5000	9-7-9	None	None	None	20	None
Std.[a]	7000	9-7-9	44 lb.	120	Dowels	30	Dowels
2-R	2500	9-7-9	None	800	Dowels	20	None
3-R	2500	9-7-9	None	400	Dowels	20	None
4-R	1500	9-7-9	None	120	Dowels	20	None
5-R	1500	9-7-9	None	120	Dowels	20	Dowels
6-R	1500	9-7-9	70 lb.	60 alt.	Dowels	60 alt.	Dowels
7-R	1200	7-7-7	None	120	None	20	None

R - Repeat Sections. Section No. 1 was not repeated.
[a] See Summary.

TABLE 2

TEMPERATURE AND PRECIPITATION DATA
July 1940 to July 1950[a]

Month	Temperature					Precipitation	
	Average	Average of the Mixima	Absolute Maximum	Average of the Minimum	Absolute Minimum	Average	Snowfall Average
	deg. F	deg. F	deg. F	deg. F	deg. F	in.	in.
December	37	47	72	28	- 6	3.0	1.5
January	34	44	76	25	-15	4.6	3.2
February	36	46	72	25	0	3.7	2.6
Winter	36	46	76	26	-15	3.8	7.3
March	47	58	85	36	0	5.7	1.6
April	57	70	90	45	25	4.0	0
May	66	78	94	54	33	3.7	0
Spring	57	69	94	45	0	4.5	1.6
June	75	87	107	63	46	4.2	0
July	77	90	103	65	44	3.5	0
August	77	90	105	64	42	3.3	0
Summer	76	89	107	64	42	3.7	0
September	67	83	99	56	32	3.6	0
October	60	72	92	46	21	2.5	0
November	46	57	84	36	- 7	4.1	0
Fall	58	71	99	46	- 7	3.4	0
Annual	57	69	107	45	-15	45.9	8.9

[a] From Special Observer Station, U. S. Weather Bureau, ½ mi. west of Owensboro, Daviess County, Kentucky.

TABLE 3

AVERAGE DAILY TRAFFIC

	1940	1941	1942	1943	1944	1945	1946	1947	1948	1949	1950
Passenger Cars	511	584	207	264	290	360	590	670	675	681	895
Light Trucks (under $1\frac{1}{2}$ tons)	150	237	413	300	325	333	363	300	282	194	255
Medium Trucks ($1\frac{1}{2}$ to 5 tons)	0	0	4	64	63	29	7	61	149	175	230
Tractor Semi-Trailers (over 5 tons)	6	8	8	11	12	16	28	22	18	2	3
Busses	8	11	17	9	10	12	15	15	16	14	18
Total Traffic	675	840	649	648	700	750	1003	1068	1140	1066	1400

tively, with 77 percent of the strengths falling within 10 percent of the average.

The 34 core specimens varied in age from 41 to 80 days, with an average compression strength of 4,855 psi. High and low strengths for the cores were 6,735 and 3,245 psi. respectively, with 47 percent of the strengths being within 10 percent of the average.

Climatological Data

Temperature and precipitation data from a station near the project are listed in Table 2. These data represent the period of pavement construction and the subsequent 10-year period of observation.

Mean annual rainfall at Owensboro for this period was 45.9 inches, with precipitation in this amount being generally representative of that for the entire state.

Severe changes in temperature were not frequent despite the excessive maximum and minimum values contained in Table 2. However, there are frequent reversals from freezing to thawing temperatures, and vice versa, within a normal winter. Past calculations (4) based on air temperatures, at a station in the central part of the state indicate that a total of about 55 such reversals occur in a representative year.

Traffic

The average daily traffic count by number and type using the projects is shown in Table 3. It should be pointed out that heavy traffic has been somewhat restricted throughout practically the entire life of the pavement because of reconstruction on other sections of the same highway and the recent completion of a bridge adequate for heavy traffic on a major river farther south within the state.

JOINT WIDTHS AND PAVEMENT ELEVATIONS

Measurements of daily, seasonal, and permanent changes in width were scheduled for a representative number of joints in each of the sections, and in addition there were five sets of precise measurements of elevation taken during the 10-year period. The number of joints represented in determinations of the average daily, seasonal, and permanent joint width measurements are noted in Table 4.

Elevation measurements were taken from points installed in a manner similar to that for the caliper inserts for width measurements, but were placed in the opposite lane. Elevation points, less frequent in number, were also installed at the midpoints of the slabs to detect warping.

Daily Measurements

With very few exceptions, the joint movement was quite uniform for all joints of a given type in a section for each date. This takes into account the fact that expansion joints and contraction joints were treated separately, in recognition of the fundamental

TABLE 4

NUMBER OF JOINTS SELECTED FOR WIDTH MEASUREMENTS

Section No.	Joint Width Measurements					
	Daily		Seasonal		Permanent	
	Exp.	Contr.	Exp.	Contr.	Exp.	Contr.
7	2	5	4	10	2	5
6	3	2	6	5	4	3
5	2	5	4	10	2	5
4	0[a]	0[a]	4	10	0[a]	0[a]
3	2	5	3	10	0	7
2	2	8	2	20	2	14
1	0[b]	8	0[b]	21	0[b]	7
Standard	3	6	5	24	3	6

[a] No measurement scheduled.
[b] No expansion joints within the section.

differences between the two.

Daily movements of the expansion joints (Fig. 1) were somewhat erratic for the individual sections, and were even more at variance by comparison among different sections. The unit movement of expansion joints generally was greater than that of contraction joints in those sections with the 120-foot intervals between expansion joints. The opposite was true in those sections where this interval was 400 feet or greater. This inconsistency is probably due to permanent closure whereby expansion joints in those sections with the long joint interval reach a "permanent" condition early and retained that closure throughout the intervening years. In fact, these joint widths showed practically no change by seasons (Fig. 2 and 2A), whereas, joint widths in all sections with shorter intervals had considerable seasonal variation.

The unit change in widths of the contraction joints as shown in Figure 1A, was relatively uniform with respect to both dates measured and the different sections.

Some significance may be attached to the fact that Section 6, having the longest slab lengths, had the smallest unit change, and the next smallest average unit change occurred in the Standard Section, which was the only other section with a joint spacing greater than 20 feet. This does not, however, take into account the fact that Section 6 had by far the greatest frequency of crack development at the time of the 1950 inspection.

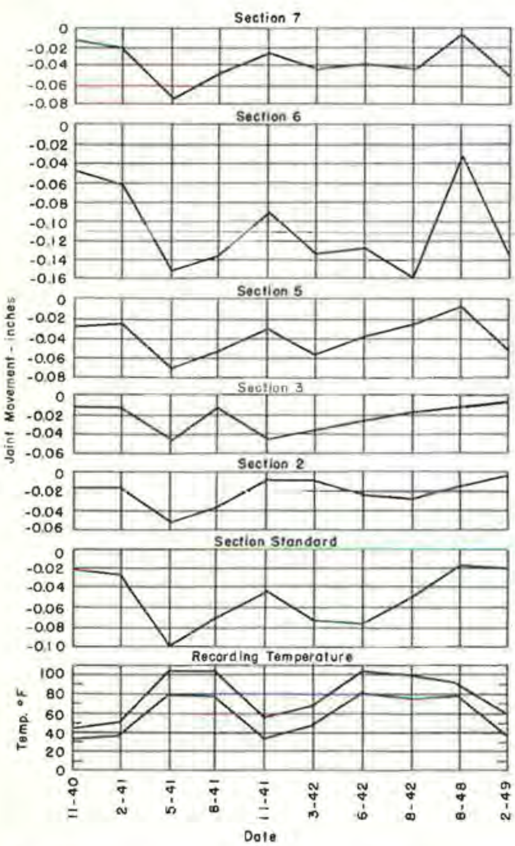


Figure 1. Average joint width change - daily. Expansion joints.

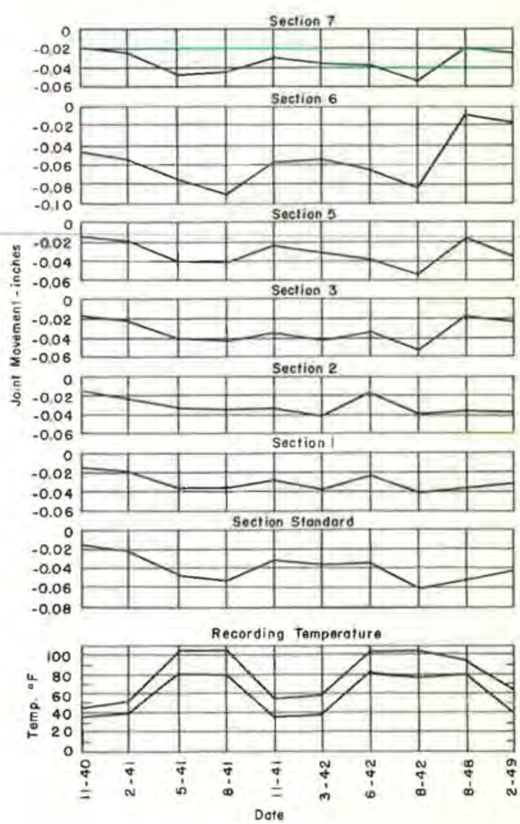


Figure 1A. Average joint width change - daily. Contraction joints.

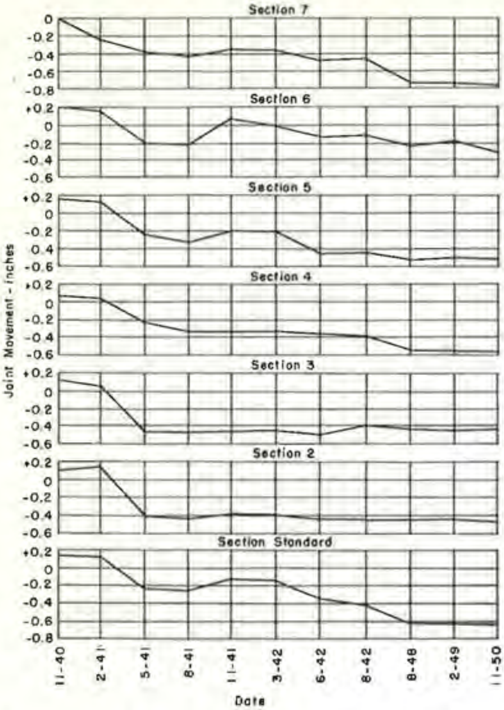


Figure 2. Average joint width change - seasonal. Expansion joints.

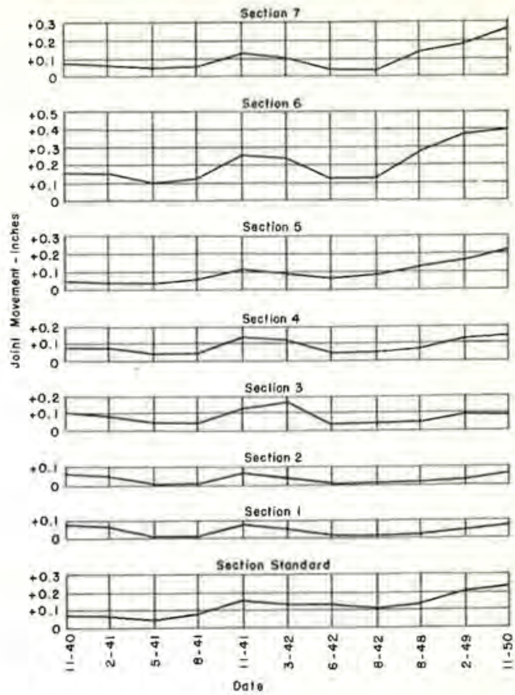


Figure 2A. Average joint width change - seasonal. Contraction joints.

Seasonal Measurements

Results of the seasonal measurements are shown in Figures 2 and 2A. The pavement showing the greatest average opening of contraction joints — and the greatest tendency on the part of those joints to remain open — was in Section 6. Contraction joints in Sections 1 and 2 almost invariably assumed their original widths at summer temperatures.

Section 6 was by far the most erratic of all from the standpoint of seasonal changes at expansion joints. For example, the expansion joints did not close and remain in "closed set" at an early age.

After the first year of service the joints were opened an average amount almost equal to the opening of the previous year. Even as late as $8\frac{1}{2}$ years after construction there was a tendency on the part of these expansion joints to show a response to seasonal conditions by opening and there were indications that this tendency would extend past the 10-year period. Actually, after the first year, contraction never approached the point of overcoming "closed set" and bringing the joints back to their original widths, but in contrast, none of the other sections showed any appreciable response to seasonal differences after 1942.

Permanent Measurements

The results of the permanent or progressive joint width measurements are shown in Figures 3 and 3A. No permanent measurements were scheduled for Section 4, and those taken for Section 6 are too erratic for evaluation. In Sections 5, 7 and Standard, the expansion joints showed a slight increase in the amount of closure each year. For the one expansion joint measured in Section 2, the closure was uniform throughout all measurements. Permanent expansion joint measurements were not included in Sections 1 and 3.

Contraction joints in Section 5 and the Standard Section in which dowel bars for load transfer were installed, remained open approximately 0. 15 inch on an average. This amount of opening was also representative of those joints in Section 7. In Sections 1, 2, and 3, the majority of contraction joints conformed to their original width measurements.

Changes in Elevation

Measurements and observations during this period intimate that changes in elevation of joints in concrete pavements do not particularly reflect or indicate structural failure in the concrete nor in its base. However, uneven settlement of these joints induces concrete deterioration which can ultimately produce failure in the pavement.

Original pavement elevations on this project were established in September, 1940. Subsequent and precise elevation measurements were made in March, 1942; July, 1944; August, 1948; and February, 1949. Table 5 shows the variations in pavement elevations from the original measurements and also the extent to which faulting has occurred in the different sections. Table 6 gives the percent of total joint faulting for all sections on specific dates. Elevations were read to the nearest 0. 005 foot by means of a standard engineer's level and leveling rod. The maximum variation in adjacent slab elevation was 0. 24 inches, which occurred at three joints, one each in Sections 1, 6, and 7.

The elevations for Section 5 taken in 1948 and 1949 were uniformly greater than those established in 1942 and 1944. This is believed to be apparent rather than actual, due to an inequality in leveling. The special bench mark for that section was destroyed between the years 1944 and 1948, and the elevations taken at later dates were established from a construction bench mark.

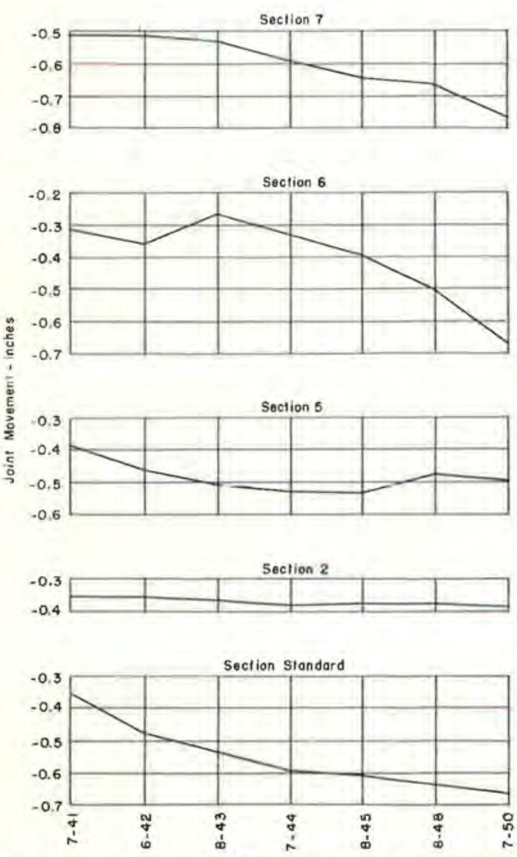


Figure 3. Average joint width change - permanent. Expansion joints.

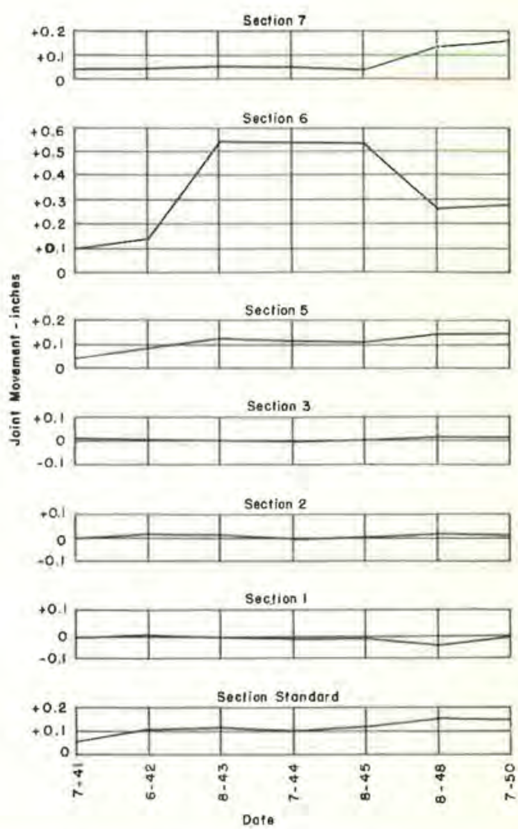


Figure 3A. Average joint width change - permanent. Contraction joints.

TABLE 5

DIFFERENCE IN ELEVATION FROM ORIGINAL ELEVATIONS

Section No. (Joints)	Measurement Date	Change in Elevation - in.		No. Joints Faulting	
		Maximum	Average	0. 12 in.	0. 24 in.
	March, 1942	0. 36	0. 17	1	0
7	July, 1944	0. 36	0. 17	4	0
(31)	August, 1948	0. 66	0. 33	6	0
	February, 1949	0. 72	0. 25	7	1
	March, 1942	0. 40	0. 23	0	0
6	July, 1944	0. 48	0. 25	0	0
(11)	August, 1948	0. 60	0. 34	1	0
	February, 1949	0. 72	0. 29	2	1
	March, 1942	0. 48	0. 27	0	0
5	July, 1944	0. 42	0. 22	2	0
(31)	August, 1948	1. 14	0. 90	3	0
	February, 1949	1. 26	1. 01	5	0
	March, 1942	0. 54	0. 30	1	0
4	July, 1944	0. 84	0. 23	2	0
(31)	August, 1948	1. 08	0. 41	1	0
	February, 1949	0. 96	0. 32	3	0
	March, 1942	0. 66	0. 40	5	0
3	July, 1944	0. 48	0. 25	2	0
(41)	August, 1948	0. 90	0. 46	5	0
	February, 1949	0. 66	0. 31	7	0
	March, 1942	0. 84	0. 38	2	0
2	July, 1944	0. 72	0. 25	4	0
(41)	August, 1948	1. 14	0. 48	6	0
	February, 1949	0. 66	0. 16	8	0
	March, 1942	0. 90	0. 53	2	0
1	July, 1944	0. 60	0. 40	3	0
(31)	August, 1948	0. 96	0. 70	0	1
	February, 1949	0. 78	0. 48	3	0
	March, 1942	0. 60	0. 30	4	0
Std.	July, 1944	0. 60	0. 24	0	0
(41)	August, 1948	1. 02	0. 43	3	0
	February, 1949	0. 78	0. 32	5	0
	March, 1942	0. 90	0. 32	15	0
Total	July, 1944	0. 84	0. 25	17	0
(258)	August, 1948	1. 14	0. 45	25	1
	February, 1949	1. 26	0. 39	40	2

Note: 0. 24 in. maximum difference observed.

TABLE 6

PERCENT OF TOTAL JOINTS FAULTING

Measurement Date	Amount of Faulting				
	0 in.	0. 06 in.	0. 12 in.	0. 18 in.	0. 24 in.
	%	%	%	%	%
March, 1942	53. 10	41. 09	5. 81	0	0
July, 1944	49. 22	44. 19	6. 59	0	0
August, 1948	47. 67	40. 31	9. 69	1. 94	0. 39
February, 1949	40. 70	41. 86	15. 50	1. 16	0. 78

PAVEMENT CONDITION

Condition surveys were conducted and reported twice yearly through 1945 and resumed again in the summer of 1948 with the latest survey being made in November, 1950. Service characteristics of the pavement at that time were generally considered satisfactory from the standpoint of

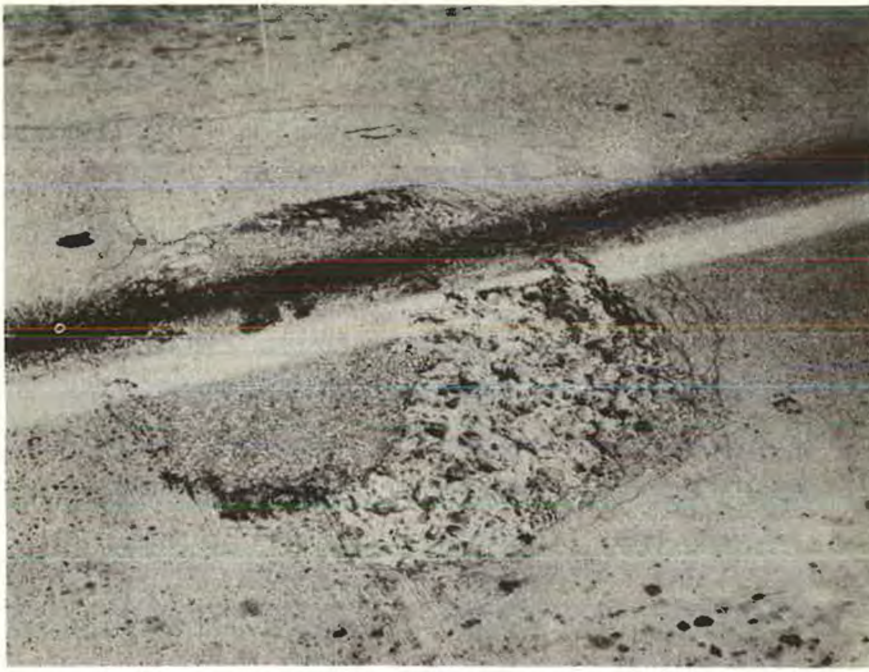


Figure 4. One of the more serious examples of spalling, Section 1 in May 1949.

Figure 5. View to north of Section 2-R through Section 1 in May 1949. Pavement in foreground contains expansion joints at 800 foot intervals with dowel bars and contraction joints are at 200 foot intervals without dowel bars. Wire mesh reinforcement was not used in this section.

Figure 6. Typical of cracking where drop inlet is constructed near joint installation, Section 4-R in May 1949.

Figure 7. Faulted joint in foreground, Section 4-R. Pavement contained expansion joints at 120 foot spacing with dowel bars and contraction joints at 20 feet without dowel bars. Wire mesh reinforcement was not installed in this section.

TABLE 7

CRACK SUMMARY BY TYPE PER SECTION

Section No.	Length (feet)	No. of Transverse Cracks		No. of Longitudinal Cracks		No. of Outside Corner Breaks		No. of Inside Corner Breaks		No. Spalling Joints
		Per Station	Per Mile	Per Station	Per Mile	Per Station	Per Mile	Per Station	Per Mile	
7	1250	10	42.4	10	42.4	7	29.1	9	38.0	1
6	1500	30	105.6	0	0.0	3	10.6	0	0.0	1
5	1500	6	21.4	1	3.6	1	3.6	0	0.0	0
4	1500	19	67.0	12	42.3	1	3.5	4	14.1	0
3	2500	5	10.6	4	8.5	0[a]	0.0	1	2.1	0
2	3000	1	1.8	11	19.4	0	0.0	0	0.0	0
1	5000	12	12.7	2	2.1	0	0.0	2	2.1	1[b]
Std.	7000	31	23.4	4	3.0	1	0.8	6	4.5	5[c]
2-R	2500	3	6.3	14	29.6	0	0.0	0	0.0	1
3-R	2500	2	4.2	8	16.9	0	0.0	0	0.0	1
4-R	1500	6	21.1	14	49.3	0	0.0	1	3.5	2
5-R	1500	7	24.6	3	10.6	0	0.0	1	3.5	0
6-R	1500	14	49.3	1	3.5	0	0.0	0	0.0	0
7-R	1200	4	17.6	3	13.2	1	4.4	2	8.8	0

[a] 1 diagonal crack.
[b] Spalling along centerline, 2 locations.
[c] Spalling in slab.

existing traffic and particularly so with respect to initial design expectations.

Faulting and Pumping

Faulting, though not infrequent, exists in such magnitude as to defer any particular emphasis on relative merits of design or imply definite association with particular construction features. Additionally, neither the presence of expansion joints nor their spacing as compared with contraction joints, had any measurable effect on faulting or differentials in pavement elevations in adjacent slabs. Little or no significant evidence of pumping was observed to have occurred in any of the sections during the 10-year period.

Figure 8. Irregular transverse cracking in east lane of Section 5-R in May 1949. Section does not contain wire mesh reinforcement.

Cracking, Corner Breaks, and Joint Deterioration

Observations throughout the test project, as disclosed in Table 7, indicate that cracking has occurred with greater frequency in the test sections than in their corresponding "repeat" sections, with the exception of the majority of longitudinal cracks.

In Sections 7 and 7-R, cracks of all types were somewhat equally represented. In Sections 6 and 6-R there was a predominance of transverse cracks which may be a consequence of greater slab lengths. However, the transverse crack interval for this section which is considerably lower than that of any of the other sections was about 50 feet. Transverse cracks also occurred rather frequently in Section 4, while in Section 4-R the number was comparable with a general average condition. The frequency of longitudinal cracks in Section 4-R was considerably greater than for the other sections.

Corner breaks, both inside and outside, were outstanding in Section 7. Sections 4 and 7-R were next in order with the number of inside corner breaks, and Section 6 with outside corner breaks. This type of cracking occurred quite infrequently in the remaining sections.

From the standpoint of all cracks appearing over the 10-year period, Section 1 had the best overall record. Performance of Sections 2, 3, and 3-R was very nearly the same, with Section 5 and Standard next in order. The rate of crack development generally increased in the last 4 years or following the survey in December, 1946. Some sections ran contrary to this trend, outstanding examples being Sections 5, 6, Standard and 2-R. Some of the spalling at joints reported from time to time has become obliterated by the application of joint sealer in maintenance operations.

Riding Qualities

A measure or comparison of riding qualities during this period could not be undertaken since this state does not maintain appropriate equipment for this evaluation. Nevertheless, the data indicate that warping and faulting offers no discernible variation in the riding qualities of slabs of different lengths.

SUMMARY

This experimental pavement, as viewed from the original scope and design, has brought to light several interesting and perhaps significant facts concerning slab behavior under varying conditions. Unfortunately, traffic conditions during the test period were somewhat inadequate for evaluation of design features under critical or maximum loads.

Despite these limitations, several differences among the sections have developed, and the effect of different variables can be analyzed to a considerable degree. More particularly, these observations may be listed as follows:

1. All expansion joints tended to close and retain a certain amount of closed set within 6 months after construction. Only Section 6, with the longest slab lengths, showed any reversal of this tendency.

2. Section 7 was unique with respect to progressive change toward closure of expansion joints. Expansion joints in that section started closing almost immediately, whereas expansion joints in all other sections opened a considerable amount during the first period of two to six months before beginning the progressive change to closed set.

3. With very few exceptions, changes in widths across joints were quite uniform for all joints of a given type measured within each section individually on each date. There were, however, great differences in the change for the different sections and for expansion joints as compared with contraction joints.

Spacing of Expansion Joints

4. Expansion joint spacing had no appreciable effect on the tendency of these joints to assume and retain a closed set, although the data pertaining to this were very meager.

5. The influence of temperature variation on changes in width of expansion joints was much greater when the spacing was relatively short — 120 feet or shorter — than when it was 400 feet or greater. After six months of service, the joints in sections with the larger intervals were hardly affected by temperature changes.

6. The unit movement of expansion joints with changes in temperature generally was greater than that of contraction joints on those sections with the 120 foot intervals between expansion joints. The reverse was true in those sections where this interval was 400 feet or greater.

7. In the sections with the 120 foot or shorter spacing of expansion joints, openings in contraction joints were greater and the tendency for them to remain open was greater than in sections where the expansion joint interval was at least 400 feet.

8. Expansion joint spacing or even the existence of expansion joints as compared with contraction joints had no measurable effect on faulting or differentials in pavement elevations in adjacent slabs.

9. The longer spacing of expansion joints – or omission of expansion joints – was conducive to fewer cracks of all types developing in slabs of equal length after construction.

10. Fewer transverse cracks, and on the whole fewer cracks of all types, developed in the sections with the lengthy expansion joint spacing than in sections having shorter expansion joint spacing and equal slab lengths. This applies to progressive crack development as well as the pavement condition at the end of the 10 years.

Spacing of Contraction Joints

Spacing of contraction joints can not be viewed as an entirely separate variable since load transfer and particularly mesh reinforcement were unique in the two sections having greater than normal spacing.

11. In the two sections where the contraction joint spacing was greater than 20 feet, the expansion joints showed the greatest tendency to return to their original widths with reductions in temperature. This was more pronounced in Section 5 with a 60 foot contraction joint interval than in the Standard Section with a 30-foot interval.

12. The extent of opening of contraction joints increased in approximate proportion with the increase in slab length. Joints in sections with the greatest interval (or greatest slab length) assumed and retained the largest opening regardless of changes in temperature. However, the computed unit change was smallest in Section 6, which had the greatest slab length.

13. Pavement elevations showed that the greater the slab length the greater the difference in elevation between the ends and centers of slabs where warping occurred. However, the average difference in elevation per foot of slab was about the same regardless of slab lengths. All sections had some warped slabs according to the measurements that were made. In most cases neither the amount nor the direction of warping remained constant year after year, and in many instances the warping reversed from a concave to convex shape. No general increase in tendency toward warping with increase in the years of service was recorded.

14. All sections had tilted slabs, but in Section 6 (60-foot slab lengths) the tendency was less pronounced and there were fewer instances of tilting in relation to the number of slabs than in the other sections with shorter slab lengths.

15. The data show no definite effect of contraction joint spacing on the development of cracks in the pavement. Not only were there variations among sections having equal joint spacing, but one of the two sections with an extraordinarily long interval had by far the greatest number of transverse cracks, and in contrast the other had no more than an average number of cracks of any type at the end of 10 years.

Load Transfer and Reinforcement

As in the case of slab lengths the presence or absence of mesh reinforcement can not be considered entirely as a separate variable, however, load transfer by dowels at the joints were varied enough to provide a limited basis for separate evaluation.

16. The data show no evidence of resistance on the part of dowels to the closure of contraction joints.

17. The prevalence of transverse cracks within about five feet of contraction joints in Section 4, as opposed to the almost complete absence of this condition in Section 5, indicates that dowel bars were beneficial in transferring load across contraction joints, even though the joints were open considerably. Similarly, corner breaks were more pronounced where the dowels were omitted. The same contrast does not exist between Section 4-R and Section 5-R.

18. No load transfer in either contraction or expansion joints in pavement with nor-

mal joint spacing resulted in exceptional deterioration of all types, if the influence of pavement thickness in Section 7 can be discounted.

19. In the absence of load transfer, closure of contraction joints and accompanying aggregate interlock tends to prevent development of cracks and corner breaks, as shown by the performance of Sections 1, 2, and 3 in contrast with Sections 4 and 7.

20. Aside from pronounced faulting at expansion joint No. 19 in Section 7 and contraction joint No. 23 in Section 4 (both having no dowels for load transfer), there was no noticeable effect of dowels on the tendency for displacement of adjacent slabs at expansion and contraction joints.

21. Lack of opportunities for comparison obscure the influence of mesh reinforcement on pavement performance. However, the high rate of transverse crack formation in both Section 6 and Section 6-R is strong evidence that the 70-lb. mesh failed to counteract the tendency toward cracking in slabs of 60 foot length. In contrast, the combination of 44-lb. mesh and 30-foot slab length resulted in a transverse crack interval that was about average for all sections.

Pavement Section

22. On the whole, the pavement of uniform 7-inch section had the poorest performance record of all pavement in the project. Much of this could possibly be attributed to the absence of load transfer bars at both contraction and expansion joints, for the contrast between Section 7 and Sections 4, 4-R and 7-R in pavement performance was not extreme despite the fact that Sections 4 and 4-R had a 9-7-9 section and dowels at expansion joints.

CONCLUSIONS

The observations and results accumulated from this experimental pavement, together with that from other states participating in the project, should contribute substantially toward a better understanding of future performance of concrete pavements and slab behavior under specific conditions. To what extent any of the results or trends established thus far might indicate future performance of this pavement, is of course a situation wholly dependent upon pavement age and service.

Nevertheless, the results obtained from this project, representing specific aggregate and specific construction methods, permit the following important conclusions: Expansion joints are of little benefit and are probably detrimental unless installed in at least 400-foot intervals; close intervals (at the most 30 feet) for contraction joints are preferable; dowel bars for load transference at contraction joints are of questionable value except in the case of joints that open considerably and remain open thus being deprived of any advantage that might be gained through interfacial pressure and aggregate interlock; the thickened edge pavement section is superior to that of uniform 7-inch thickness; and mesh reinforcement alone will not prevent cracking particularly in slabs greater than 30 feet in length.

With due regard to the very narrow margin for differentiation in some cases, overall performance characteristics by sections were from best to poorest in the following order: 1, 2, 3, 5, Standard, 6, 4, and 7.

ACKNOWLEDGEMENTS

During the course of this experimental project, several progress reports have been written under different authorship by men of the Kentucky Department of Highways, the latest being a 9-year report by S. T. Collier, Senior Research Engineer. Inasmuch as the tenth year data and observations have continued to substantiate the conclusion previously drawn by Collier, the bulk of this report, along with most observations, were stated previously in his report.

References

1. Kelley, E. F., "History and Scope of Cooperative Studies of Joint Spacing in Concrete," Proceedings, H. R. B., Vol. 20, December 1940.

2. Thomas, T. R., "Investigational Concrete Pavement in Kentucky," H. R. B. Re-

port No. 3B, November 1945.

3. Collier, S. T., "Cooperative Investigation of Joint Spacing in Concrete Pavements," Kentucky Department of Highways, Progress Report No. 4, Proj. FA 125 F (2)S, July 1949, (Unpublished).

4. Gregg, L. E., "Experiments with Air Entrainment in Cement Concrete," Univ. of Kentucky Engr. Exp. Sta. Bull. No. 5, September 1947.

Report on Experimental Project in Michigan

H. C. COONS,* Deputy Commissioner, Chief Engineer
Michigan State Highway Department

● 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.

*Deceased

TABLE 1

SUMMARY OF TEST AREAS

Test Area Designation: Series	Test Area Designation: Division	Number of Sections in Division	Length of Division in feet	Pavement Thickness, inches	Reinforcement lb./100 sq. ft.	Joint Spacing in feet: Expansion	Joint Spacing in feet: Contraction	Joint Spacing in feet: Dummy	Load Transfer Type: Expansion (1)	Load Transfer Type: Contraction (2)	Load Transfer Type: Dummy (3)	Filler and Seal: 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	1830	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

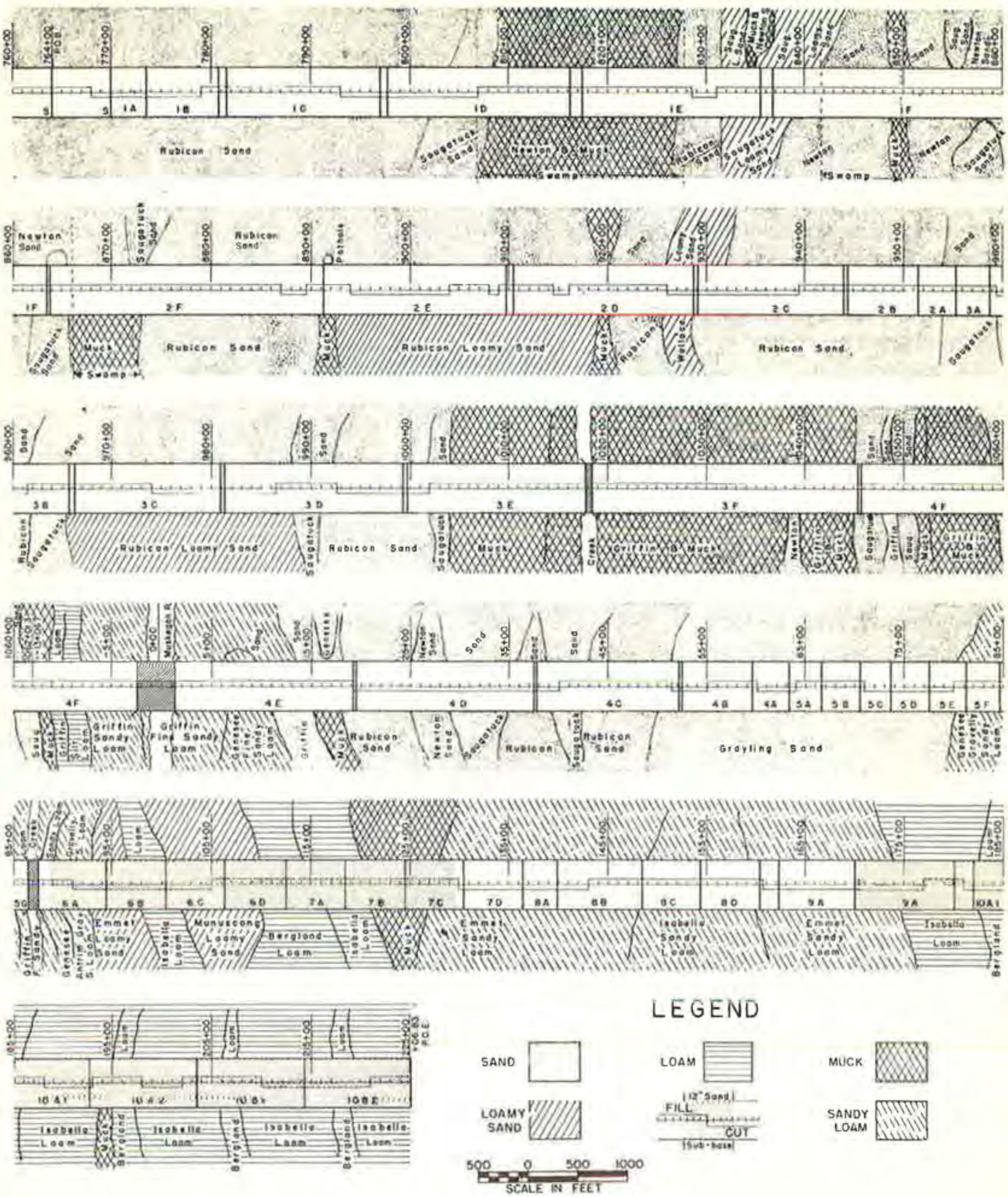


Figure 1. Soil types and earthwork operations.

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.

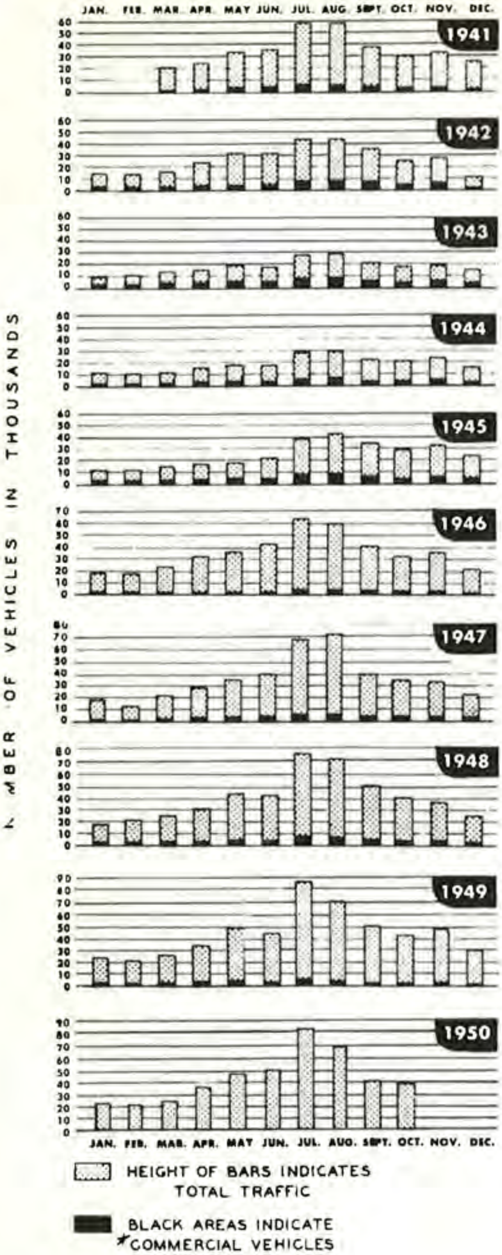


Figure 2. Monthly traffic record.

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

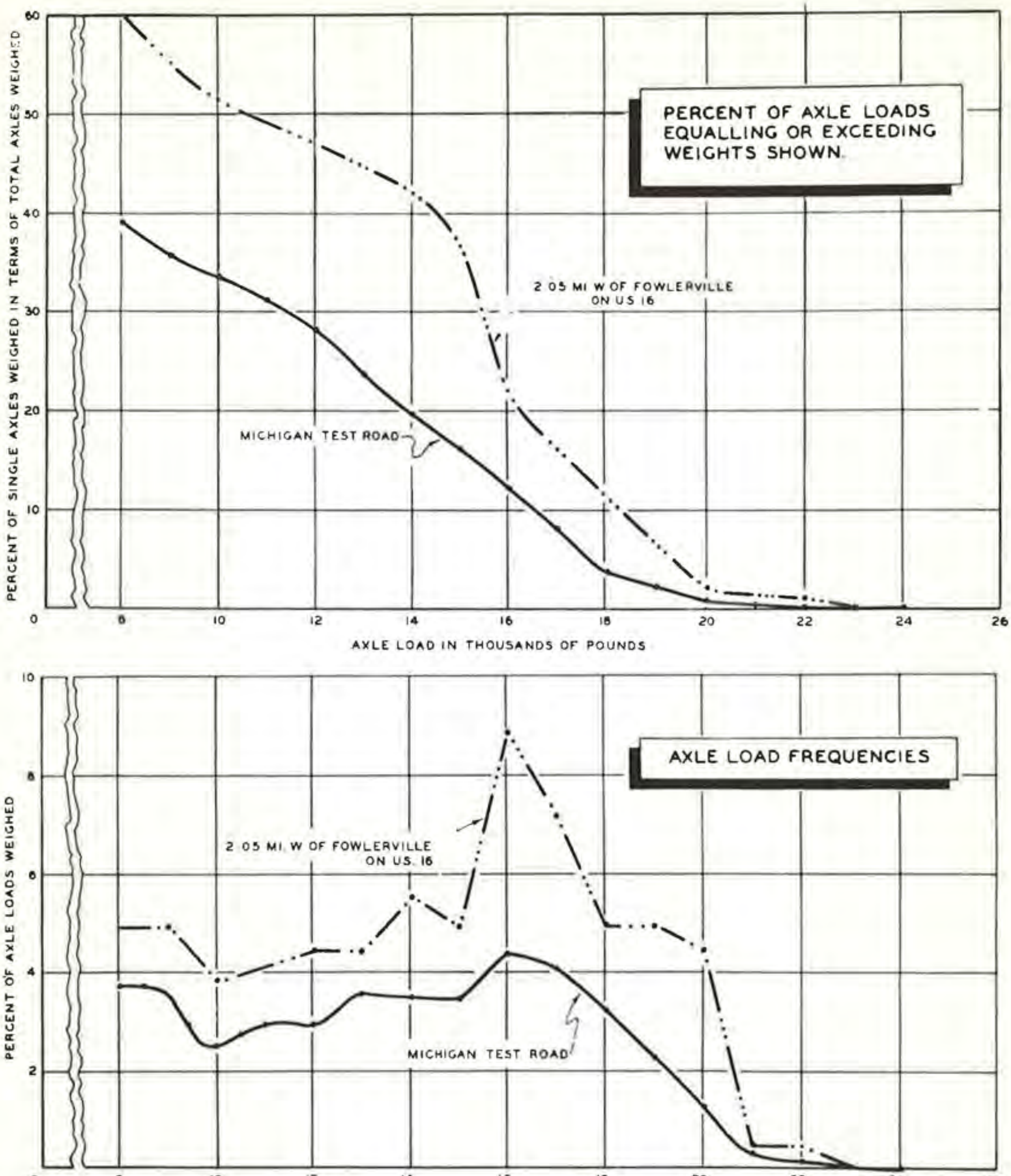


Figure 3. Comparison of axle loads on test road with primary Route US 16.

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

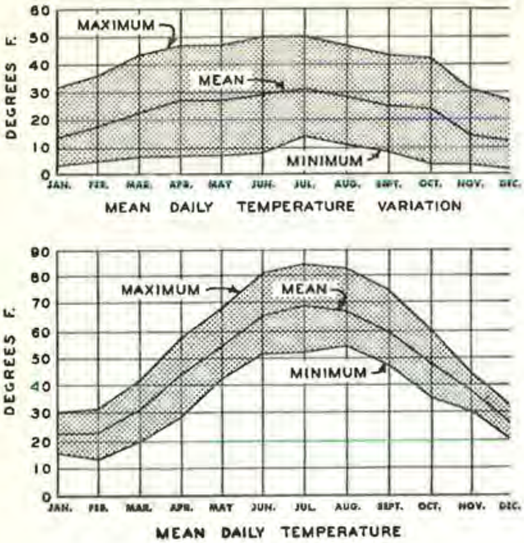


Figure 4. Temperature record.

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-

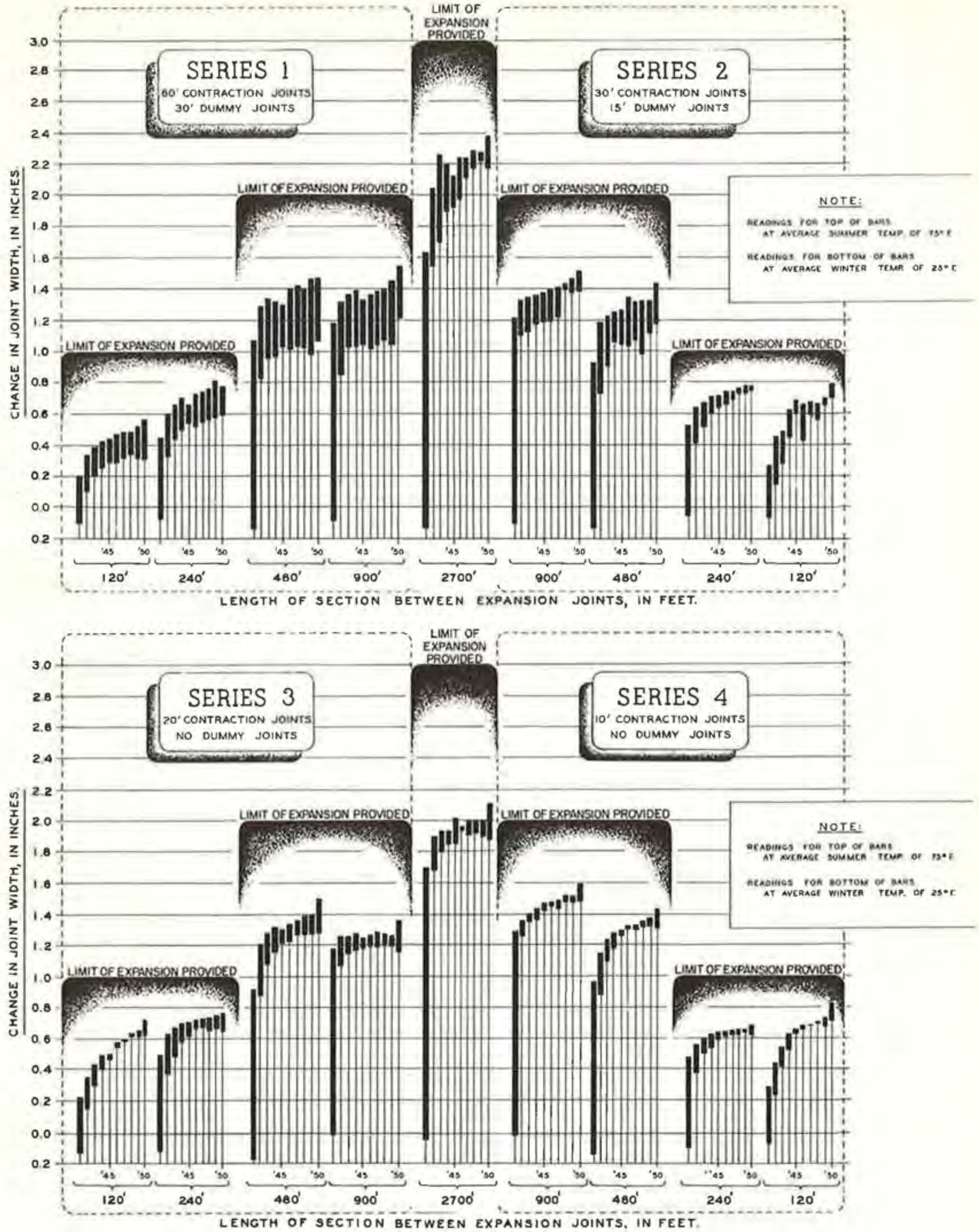


Figure 5. Annual and progressive changes in expansion joint width.

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

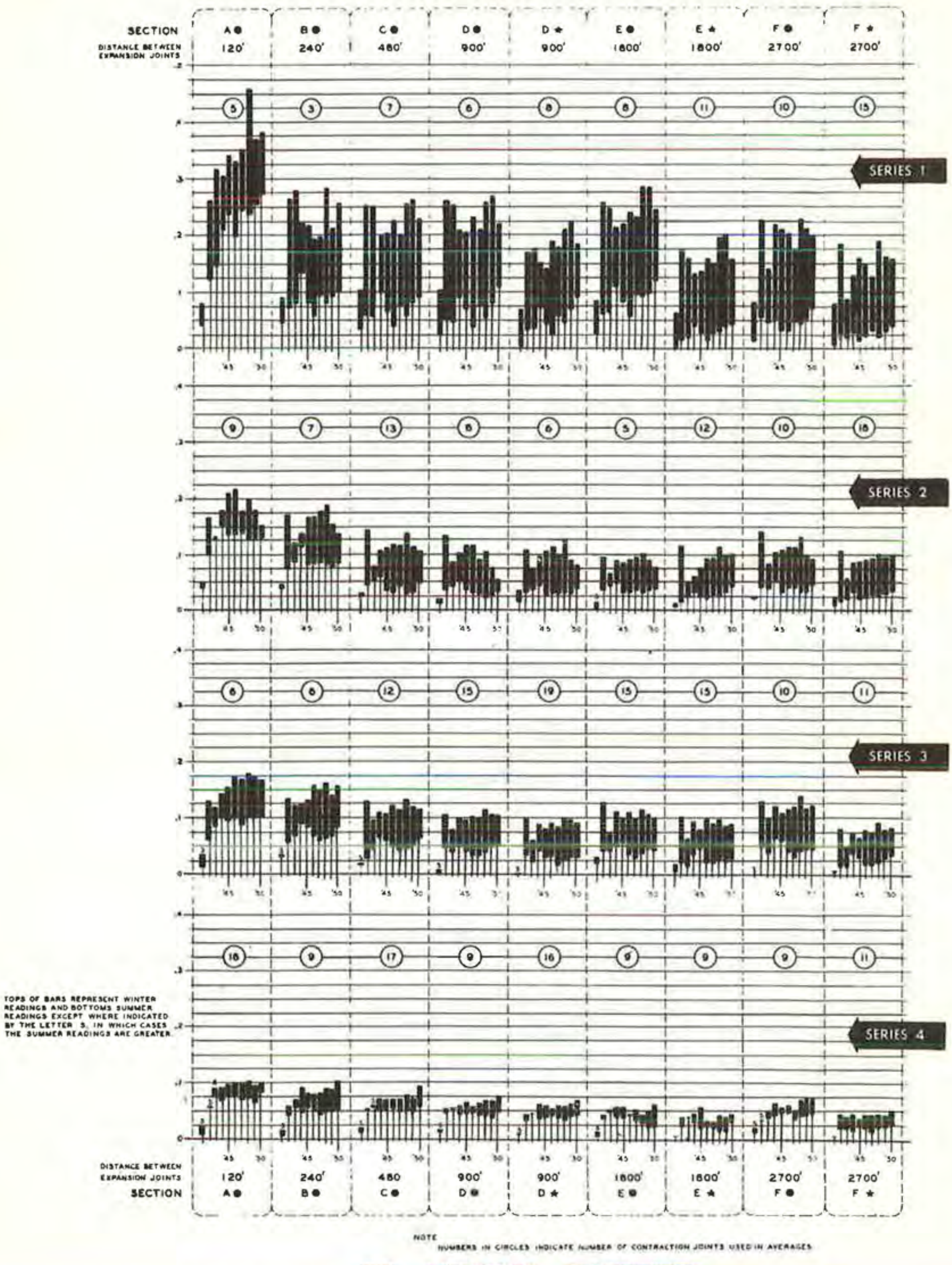


Figure 6. Seasonal changes in contraction joint width.

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.

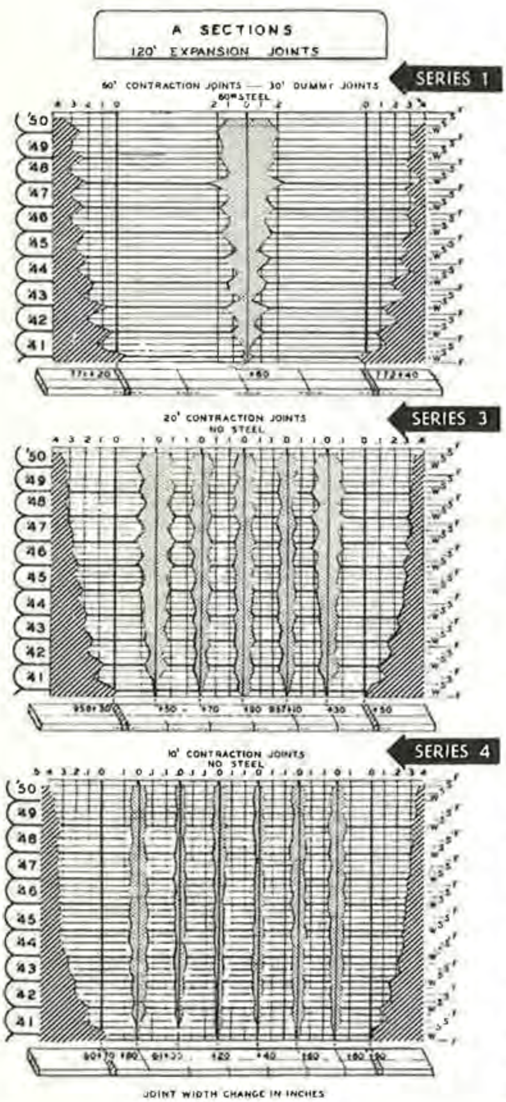


Figure 7. Contraction joint residual movement.

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

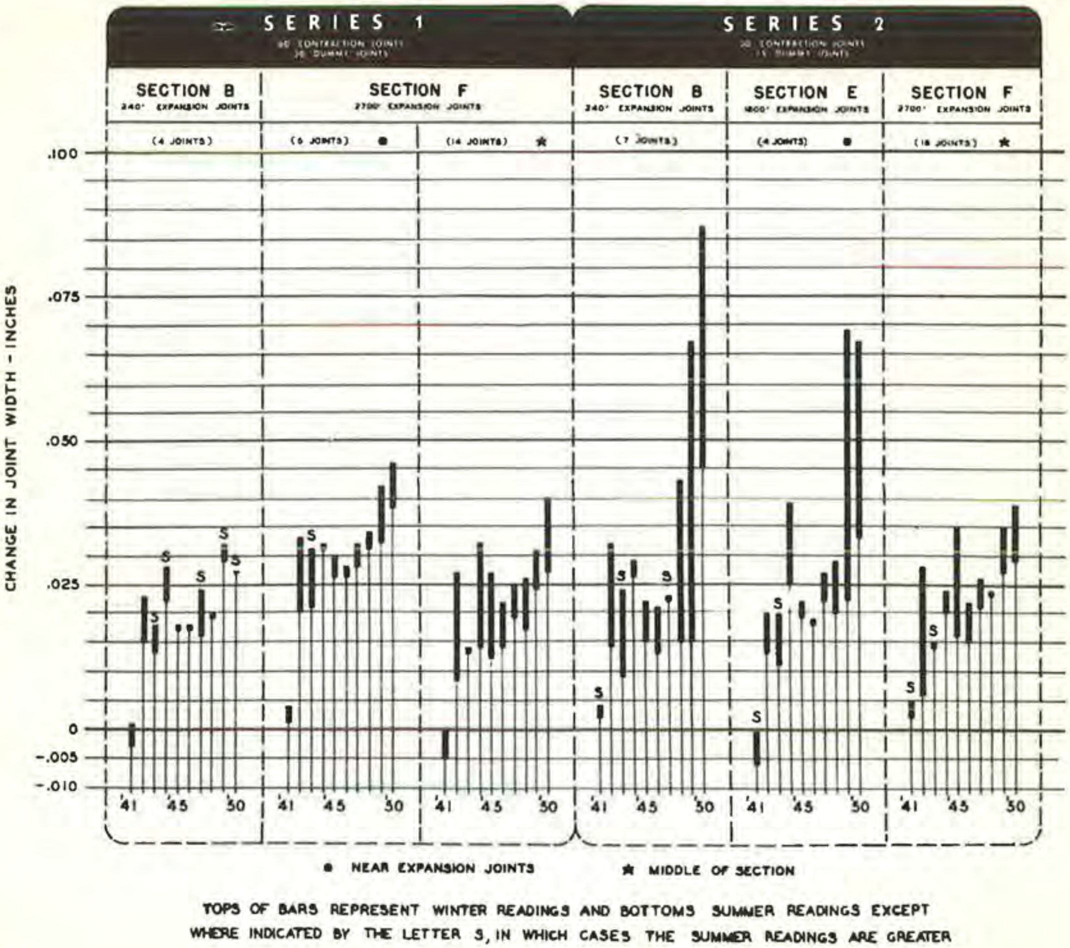


Figure 8. Seasonal changes in dummy joint width.

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

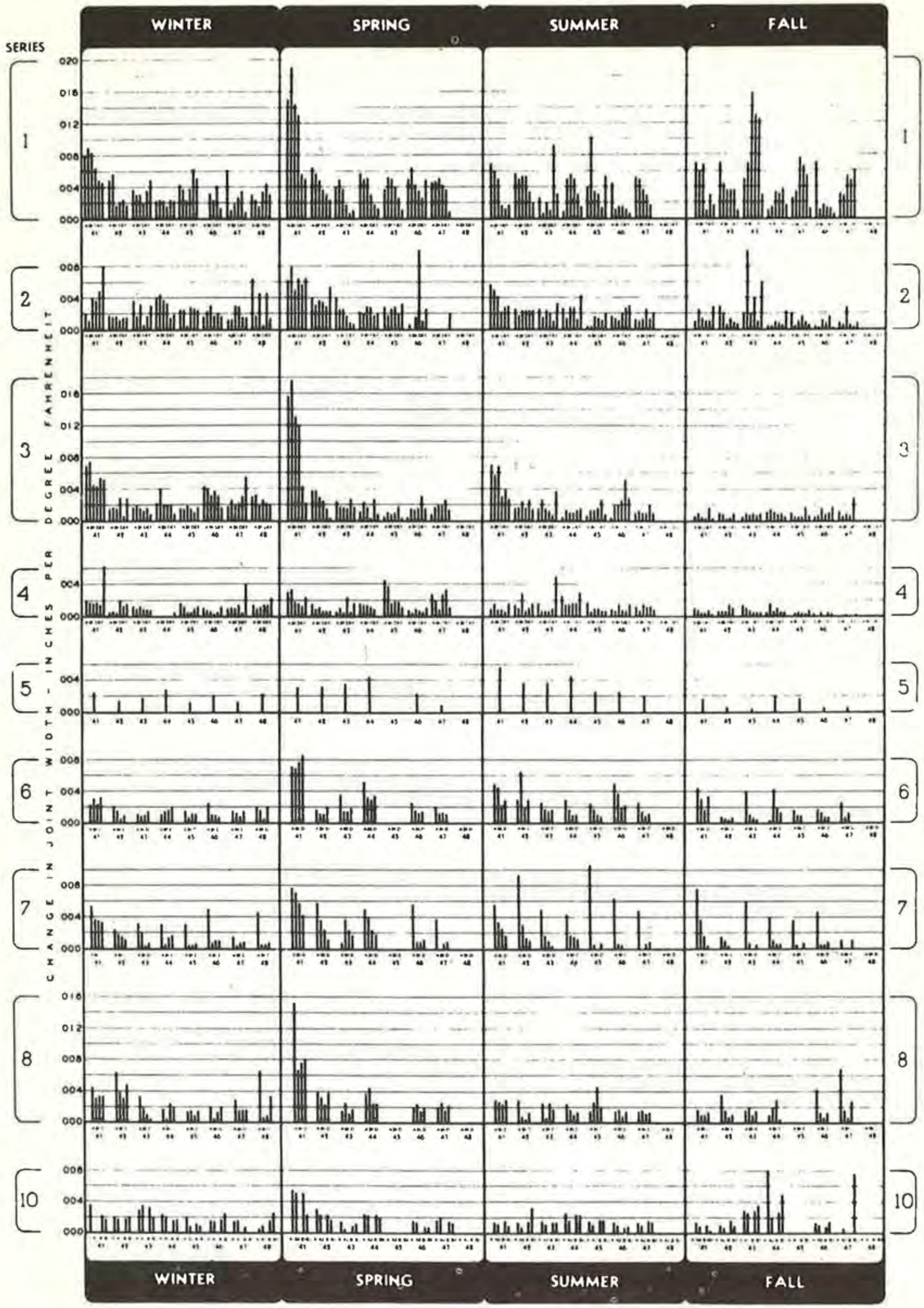


Figure 9. Daily movement of expansion joints based on average of individual joints.

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.

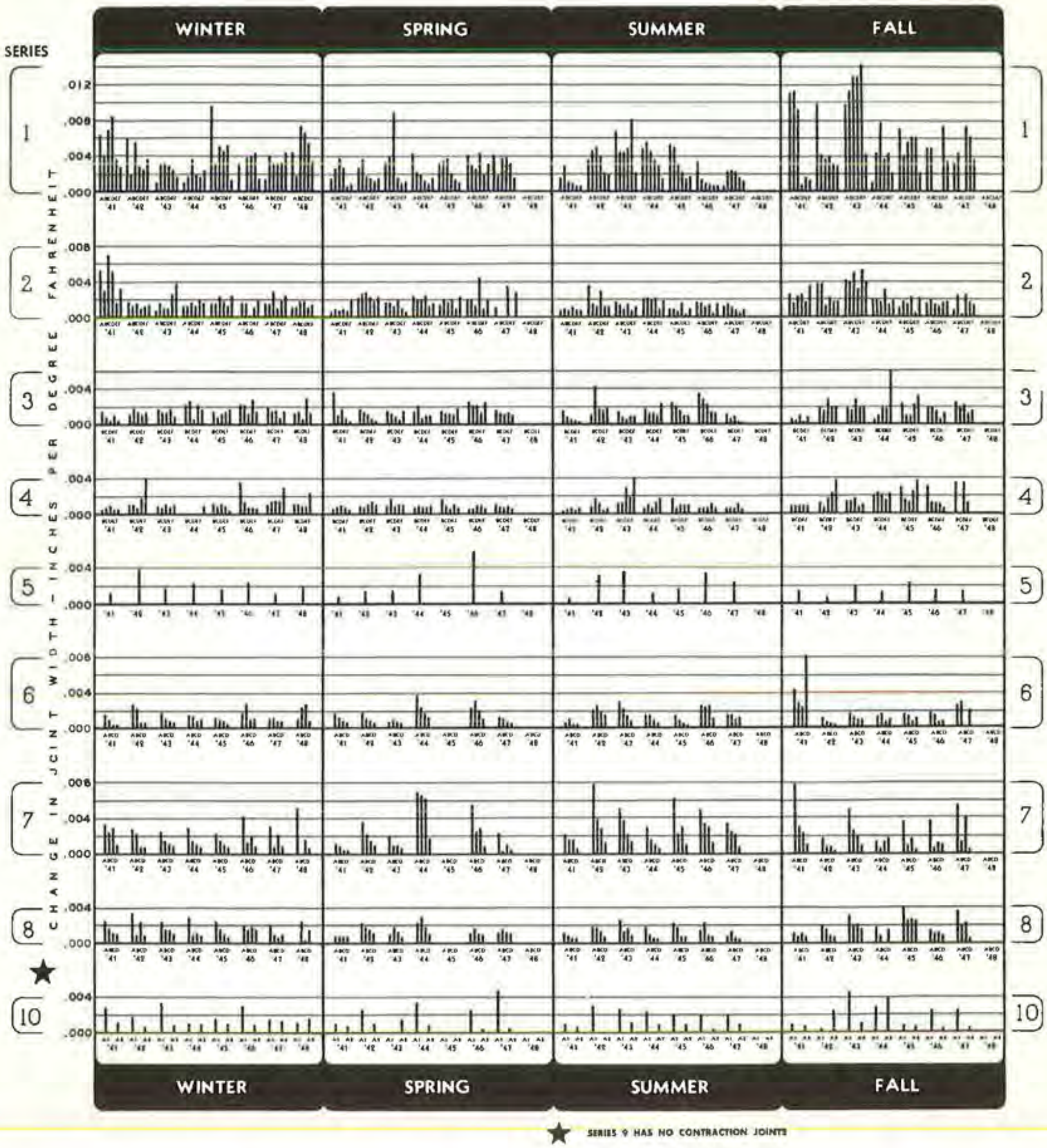


Figure 10. Daily movement of contraction joints.

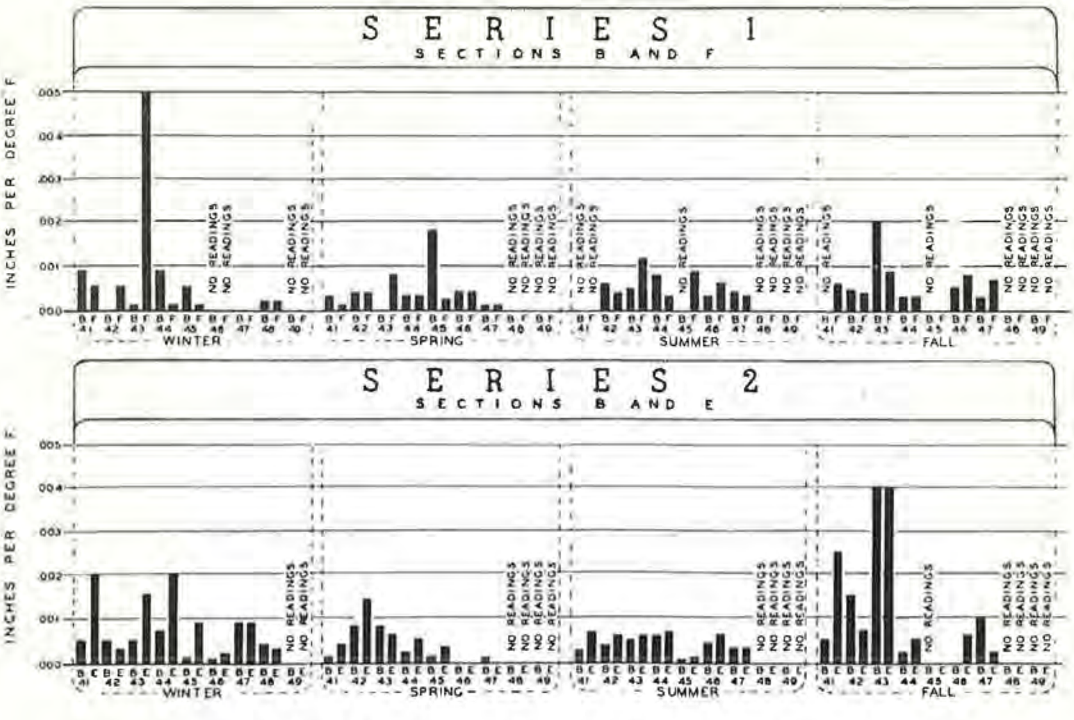


Figure 11. Daily movement of dummy joints.

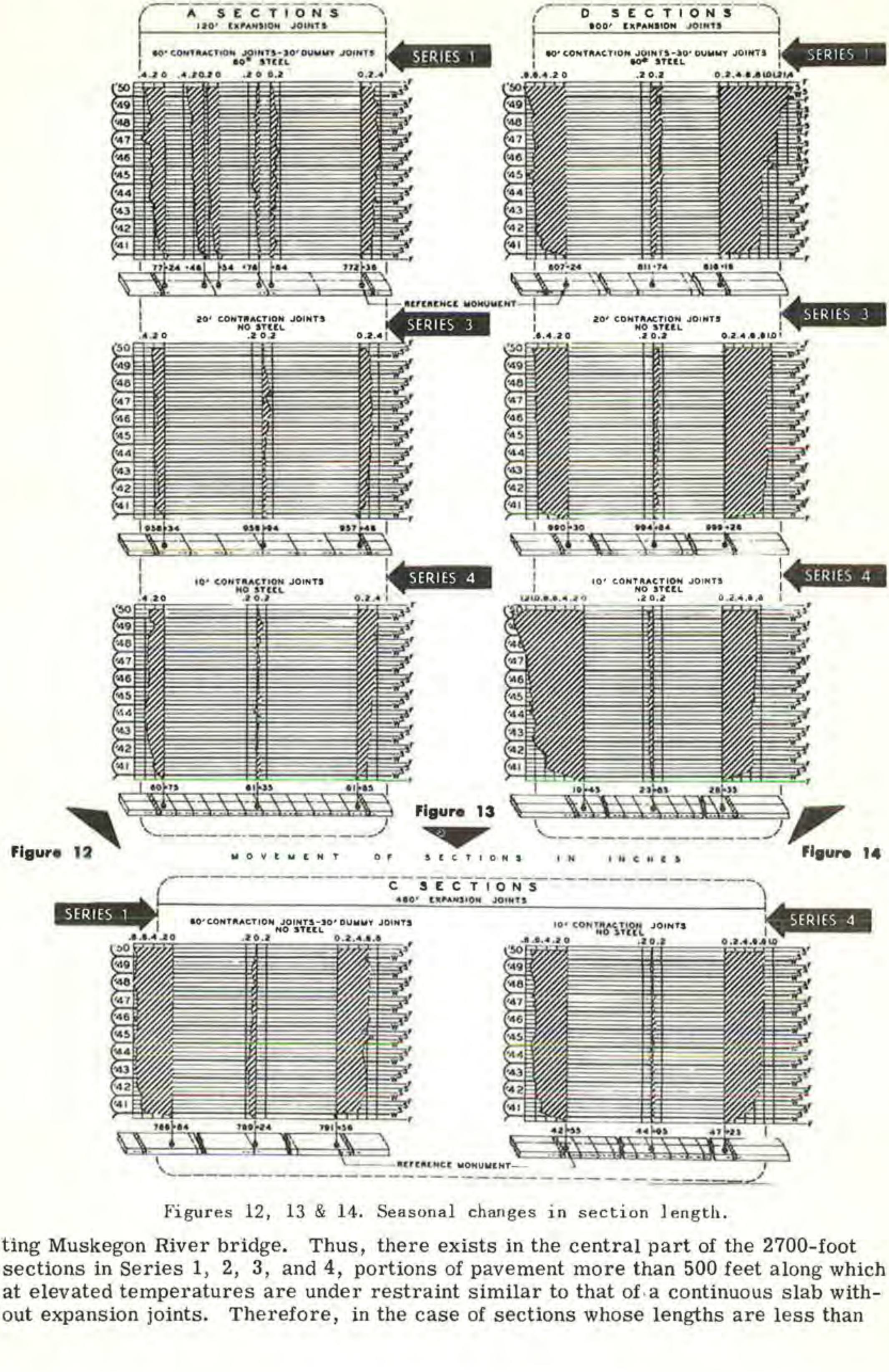
Contraction Joints. In a similar manner the average daily contraction joint width movements have been presented in Figure 10. The data presented in Figure 10 shows in general that the contraction joints behave in the same manner as the expansion joints. Because of the greater number of joints involved 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-



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

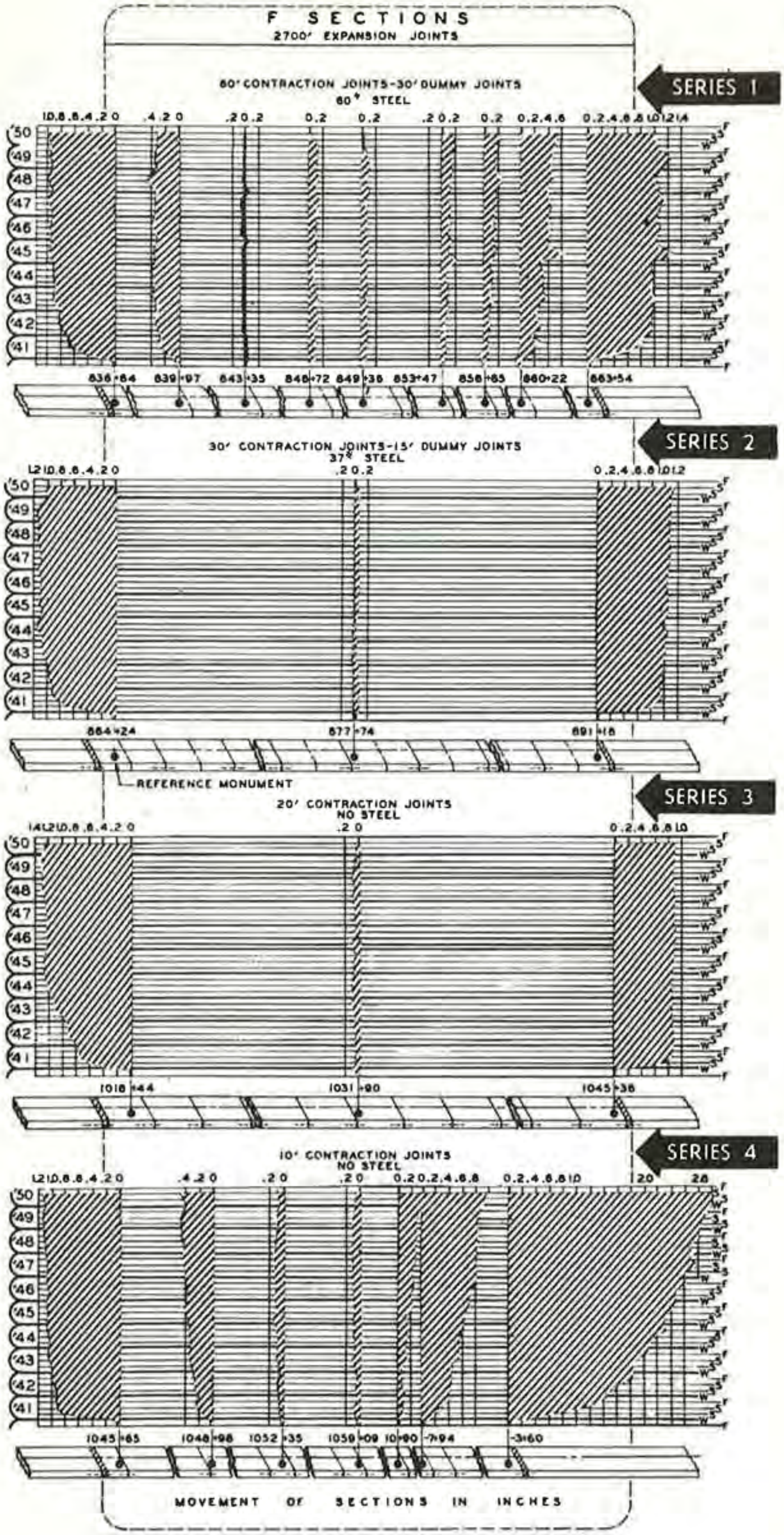


Figure 15. Seasonal changes in section length.

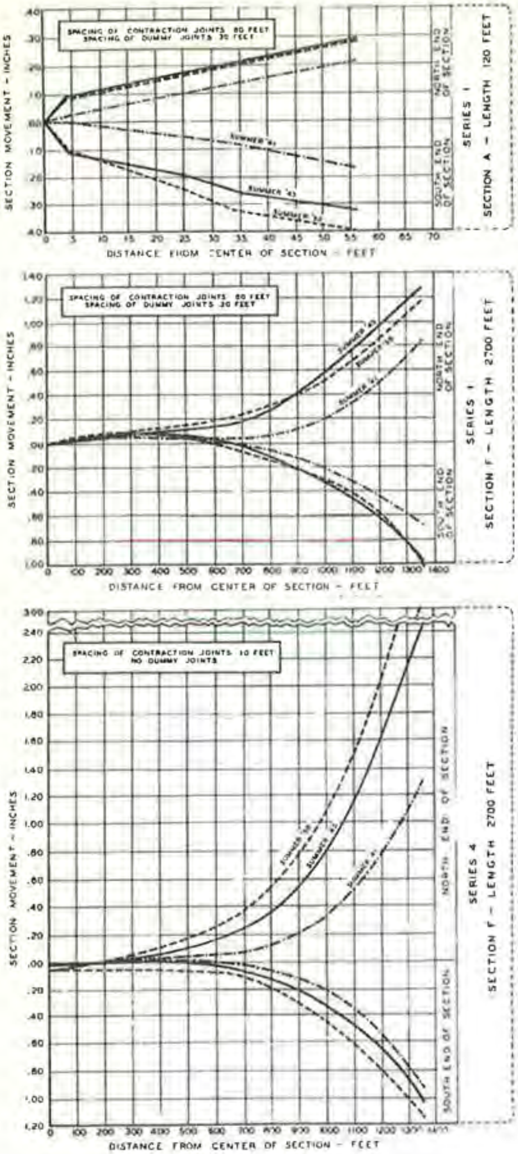


Figure 16. Relation between section movement and distance from center of section.

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, 80 lb. Steel: Total Slabs	No. Cracked	Percent Cracked	Cracking, ft.: Transverse	Diagonal	Longitudinal	Total	Series 2 (9"-7"-9") 15 Foot Slabs, 37 lb. Steel: Total Slabs	No. Cracked	Percent Cracked	Cracking, ft.: Transverse	Diagonal	Longitudinal	Total	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	12	1	8	7	0	0	7	28	0	0	0	0	0	0	18	0	0	0	0	0	0	36	0	0	0	0	0	0
B	240	24	0	0	0	0	0	0	47	0	0	0	0	0	0	36	0	0	0	0	0	0	72	0	0	0	0	0	0
C	480	48	0	0	0	0	0	0	96	0	0	0	0	0	0	72	4	8	53	0	0	53	144	0	0	0	0	0	0
D	900	61	3	5	40	0	0	40	120	3	2	66	0	0	66	95	4	4	34	0	0	34	180	0	0	0	0	0	0
E	1800	83	4	7	39	0	6	45	170	0	0	0	0	0	0	90	4	5	11	10	35	56	180	0	0	0	0	0	0
F	2700	89	13	16	167	0	0	167	175	0	0	0	0	0	0	135	3	2	30	0	0	30	270	0	0	0	0	0	0
Summary		295	23	8	253	0	6	259	486	3	1	66	0	0	66	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	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	12	0	0	0	0.0	24	[illegible]	4	1	1.0	28	1	14	4	1.0	36	1	3	1	1.0
B	240	24	6	25	7	1.1	47	[illegible]	3	1	1.0	36	2	8	2	1.0	72	5	7	5	1.0
C	480	48	[illegible]	14	11	1.7	96	[illegible]	13	18	1.4	72	20	26	72	1.1	144	12	8	15	1.2
D	900	60	13	19	28	2.0	120	[illegible]	25	32	1.3	98	16	16	17	1.1	180	8	4	10	1.2
E	1800	63	22	35	70	1.3	120	[illegible]	[illegible]	19	1.4	90	32	36	33	1.1	180	3	2	3	1.0
F	2700	90	34	39	40	1.6	170	19	11	26	1.4	125	13	10	13	1.2	270	12	4	11	1.1
Summary		294	82	28	122	1.7	598	[illegible]	13	97	1.3	451	87	19	95	1.1	882	41	5	55	1.2

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.

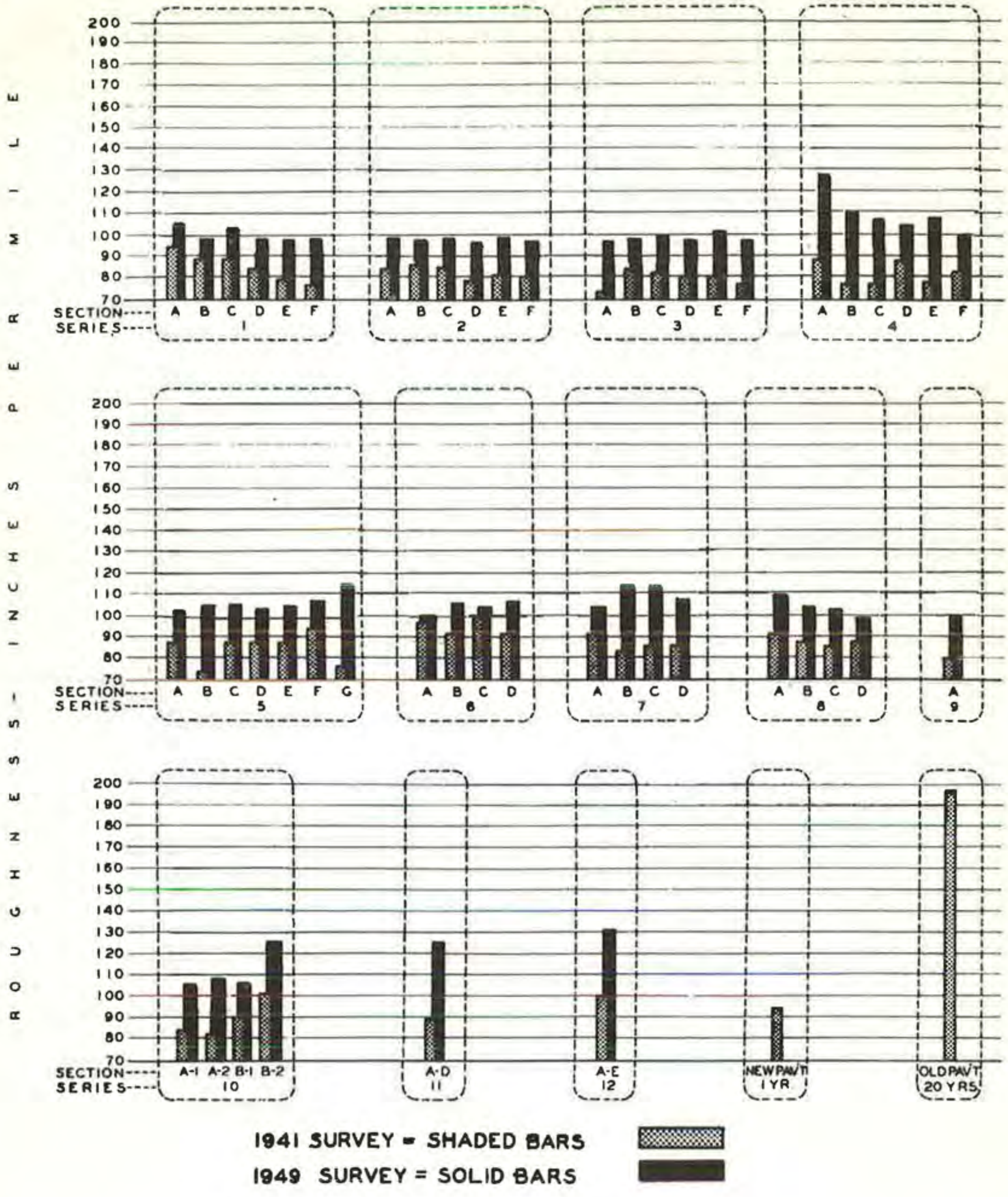


Figure 17. Road roughness data.

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

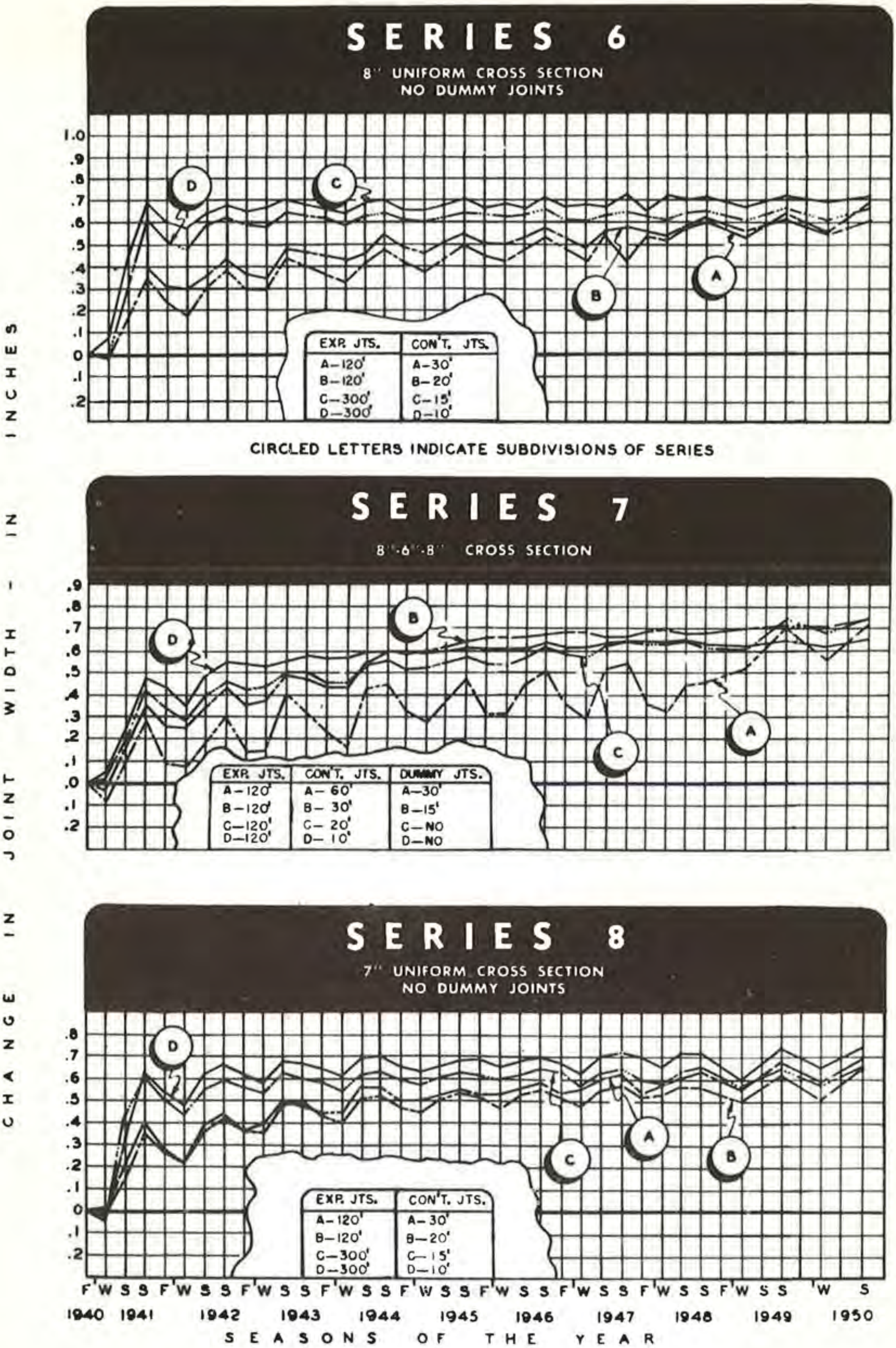
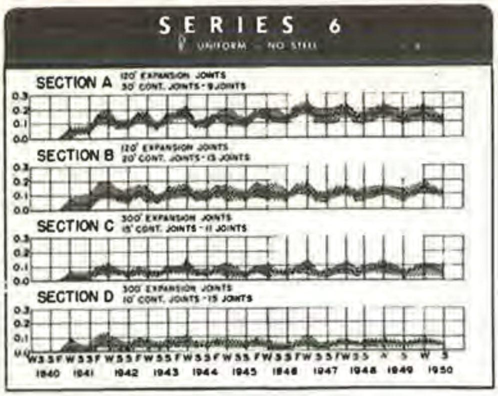
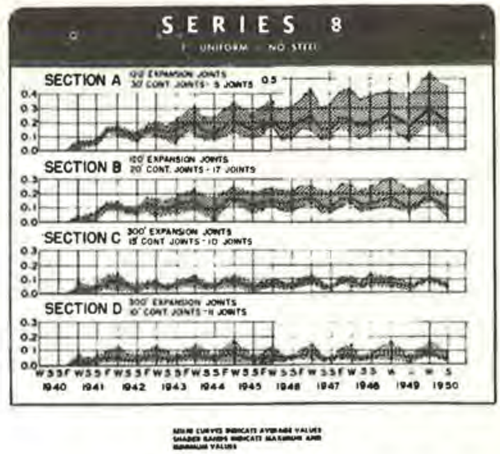


Figure 18. Seasonal changes in expansion joint width.





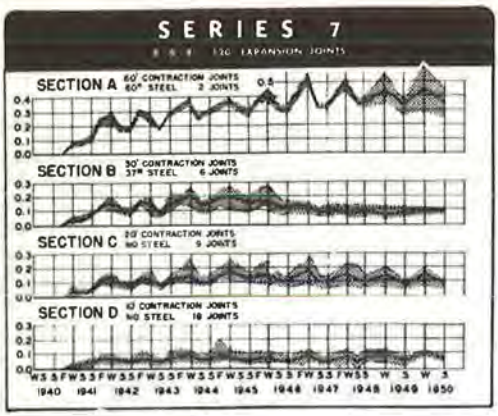


Figure 19. Seasonal changes in contraction joint width. Change in joint width, in inches.

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.

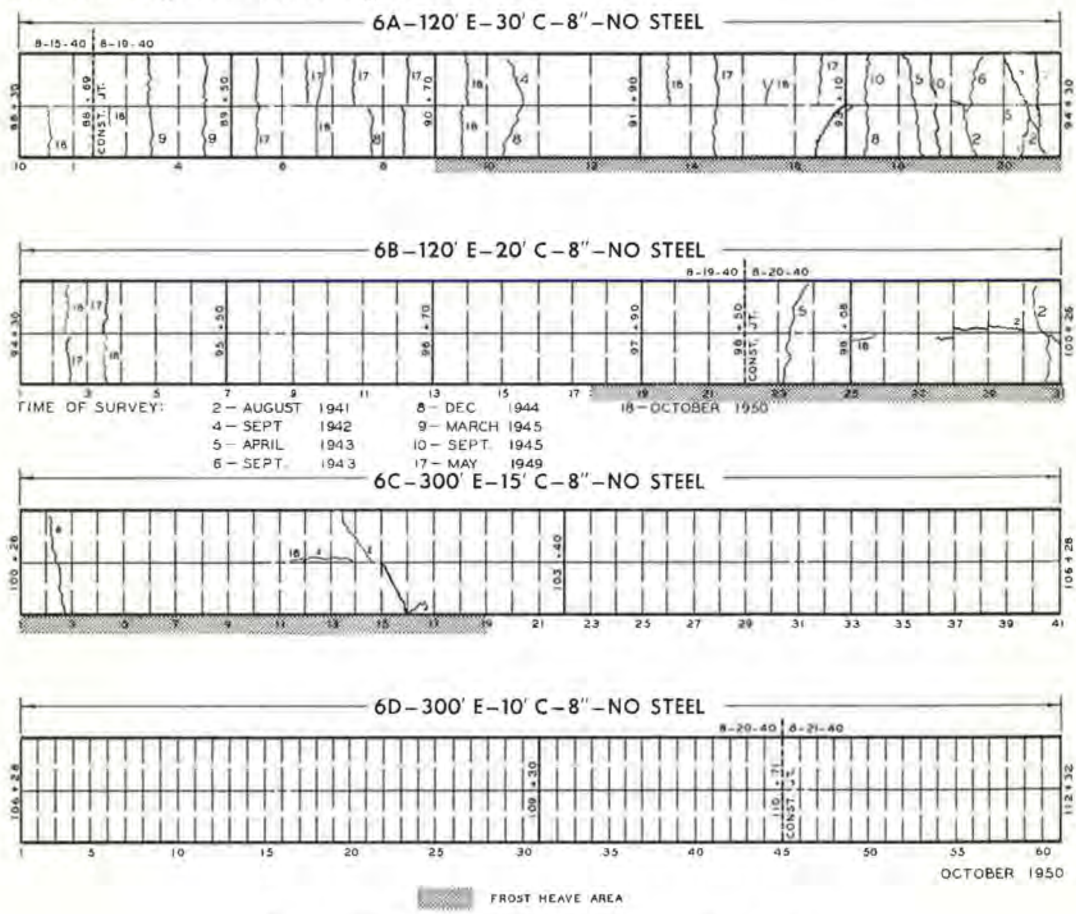


Figure 20. Condition of pavement in Series 6.

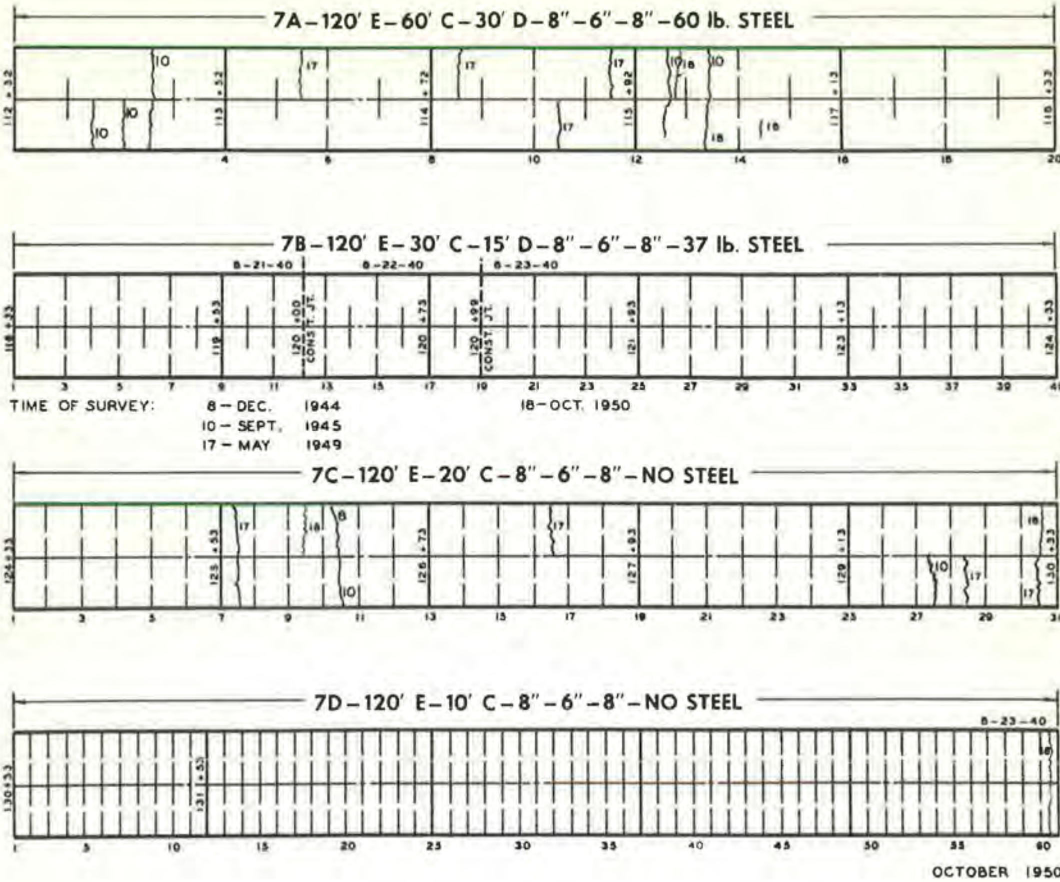


Figure 21. Condition of pavement in Series 7.

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.

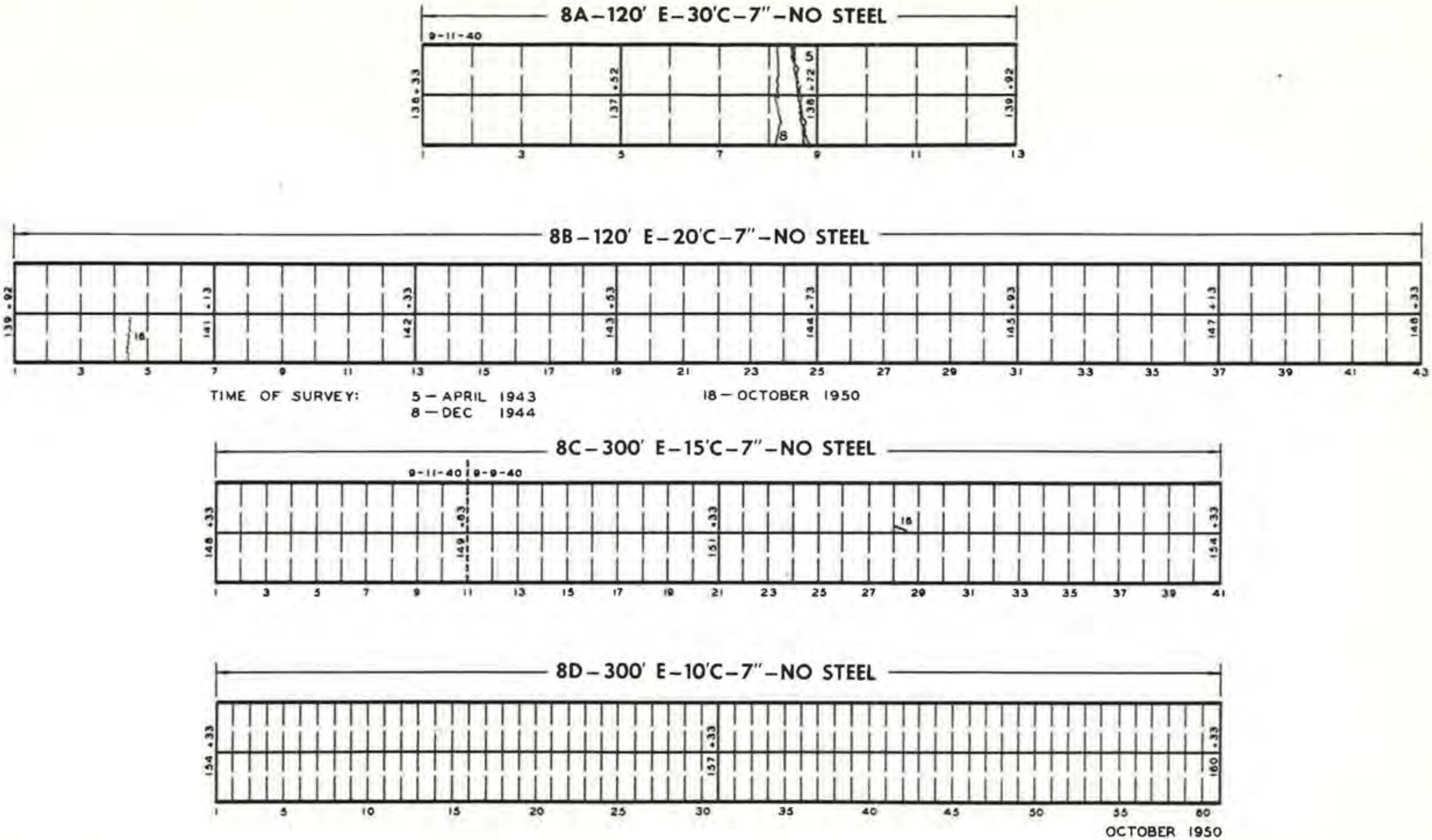


Figure 22. Condition of pavement in Series 8.

TABLE 8

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

Series 6 – 8" Uniform, No Reinforcement

Section No.	Slab Length	Exp. Joint Spacing	No. Slabs Total[a]		No. Slabs Cracked Total[a]		Percent Cracked Total[a]		Cracking, ft. Transverse Total[a]		Cracking, ft. Diagonal Total[a]		Cracking, ft. Longitudinal Total[a]		Cracking, ft. Total Cracking	Cracking, ft. Normal Cracking[a]
A	30	120	20	8	18	8	90	100	322	139	111	0	4	0	437	139
B	20	120	30	16	8	2	27	12	33	48	38	0	62	0	133	48
C	15	300	40	22	7	0	17	0	0	0	93	0	48	0	141	0
D	10	300	60	0	0	0	0	0	0	0	0	0	0	0	0	0
Summary			150	46	33	10	21	30	355	137	242	0	114	0	711	187

Series 7 – 8"-6"-8" Cross Section, Reinforcement – 60 lb. – 37 lb. – None

Section No.	Exp. Joint Spacing	No. Slabs	No. Cracked	Percent Cracked	Cracking, ft. Transverse	Cracking, ft. Diagonal	Cracking, ft. Longitudinal	Cracking, ft. Total	Reinforcing lb./100 sq. ft.
A	120	20	9	45	146	0	0	146	60
B	120	40	0	0	0	0	0	30	37
C	120	30	7	23	110	0	0	110	None
D	120	60	1	2	22	0	0	22	None
Summary		150	17	11	278	0	0	308	

Series 8 – 7" Uniform, No Reinforcement

Section No.	No. Slabs	No. Slabs Cracked	Percent Cracked	Cracking, ft. Transverse	Cracking, ft. Diagonal	Cracking, ft. Longitudinal	Cracking, ft. Total Cracking
A	12	1	8.3	22	23	0	45
B	42	1	2.4	11	0	0	11
C	40	1	2.5	0	3	0	3
D	60	0	0	0	0	0	0
Summary	154	3	2	33	26	0	59

Spalling

Section No.	Slab Length	Exp. Joint Spacing	Series 6 Number Joints	Series 6 Number Spalled	Series 6 Percent Spalled	Series 6 Number Spalled Areas	Series 6 Average per Joint	Series 7 Number Joints	Series 7 Number Spalled	Series 7 Percent Spalled	Series 7 Number Spalled Areas	Series 7 Average per Joint	Series 8 Number Joints	Series 8 Number Spalled	Series 8 Percent Spalled	Series 8 Number Spalled Areas	Series 8 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

Series	Slab Thickness	30' A Total Cracking ft.	30' A Total Slabs	30' A Percent Slab Cracks	30' A Spalled Joints %	20' B Total Cracking	20' B Total Slabs ft.	20' B Percent Slabs Cracked	20' B Spalled Joints %	15' C Total Cracking	15' C Total Slabs ft.	15' C Percent Slabs Cracked	15' C Spalled Joints %	10' D Total Cracking ft.	10' D Total Slabs	10' D Percent Slabs Cracked	10' D 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	
3-A	9"-7"-9"	-	-		-	0	18	0	14	-	-	-	-	-	-	-	-	0	
2-B	9"-7"-9"	-		-	-	-	-	-	-	0	47	0	2	-	-	-	-	0	6
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-

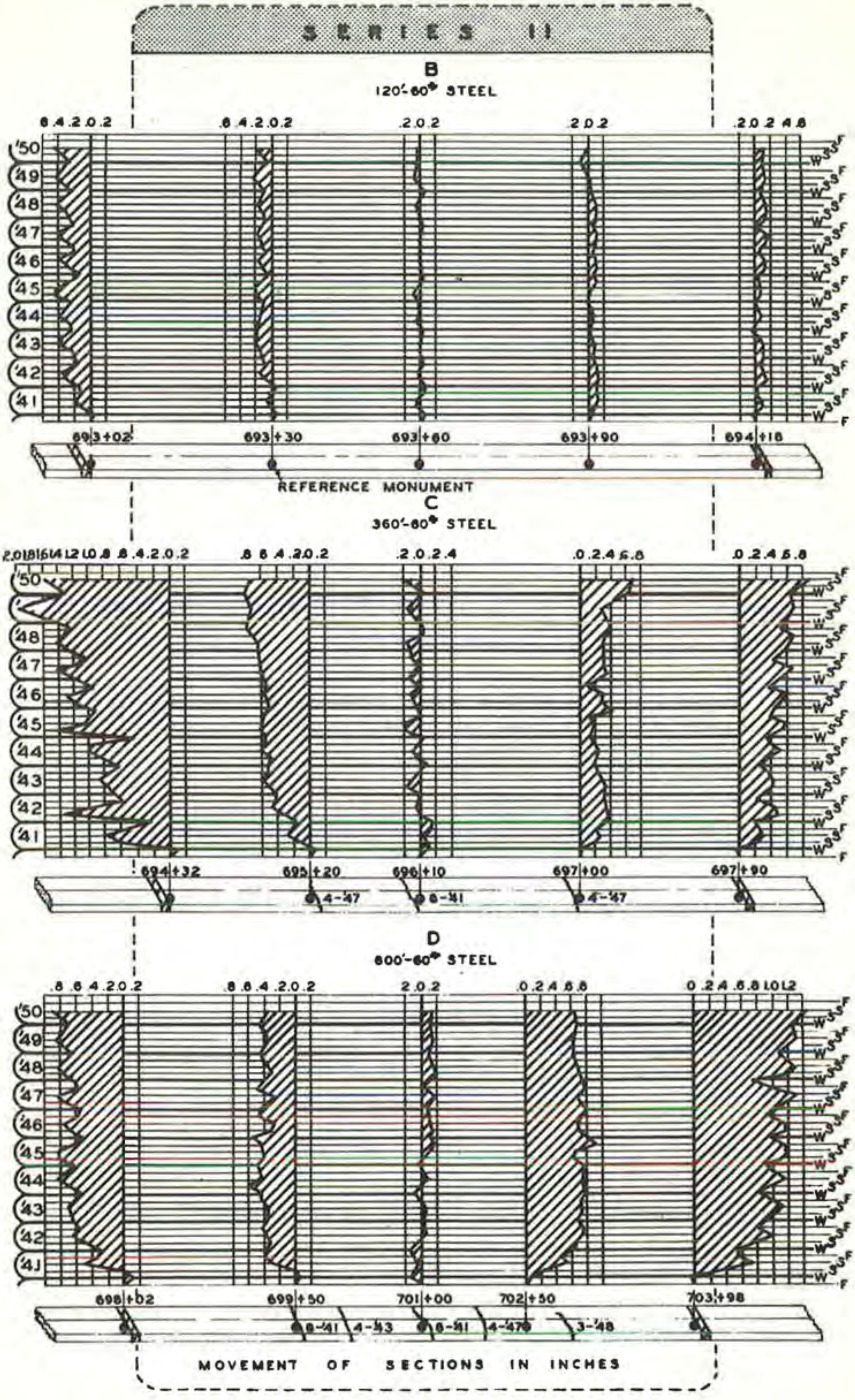


Figure 23. Seasonal changes in section length.

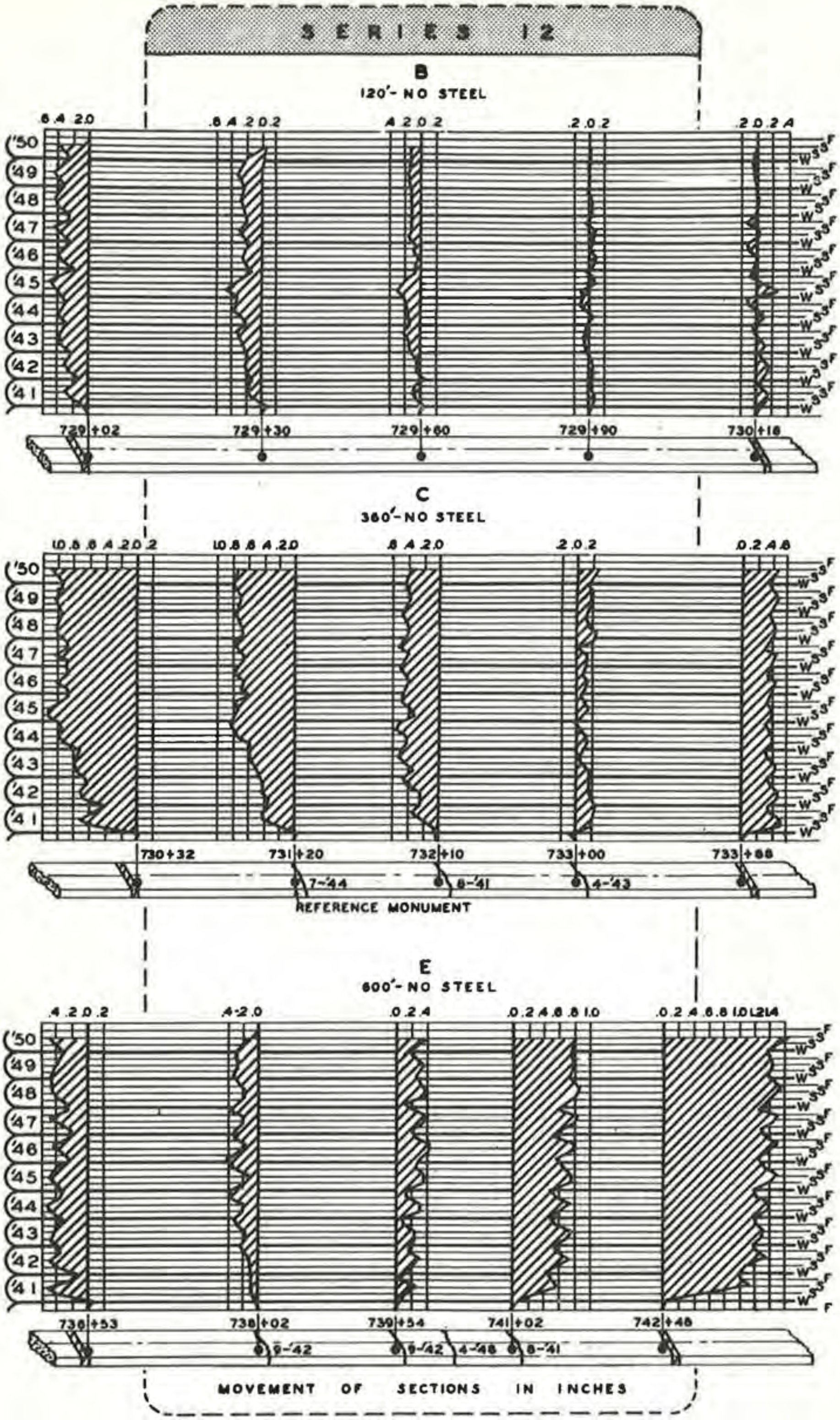


Figure 24. Seasonal changes in section length.

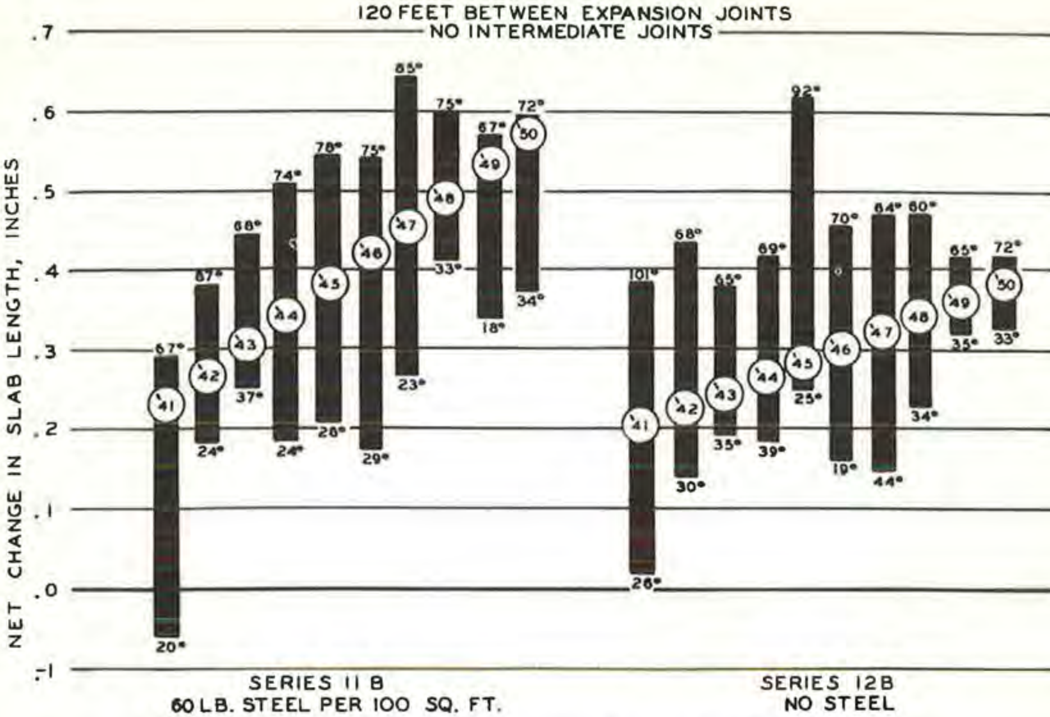


Figure 25. Annual and progressive changes in slab length.

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

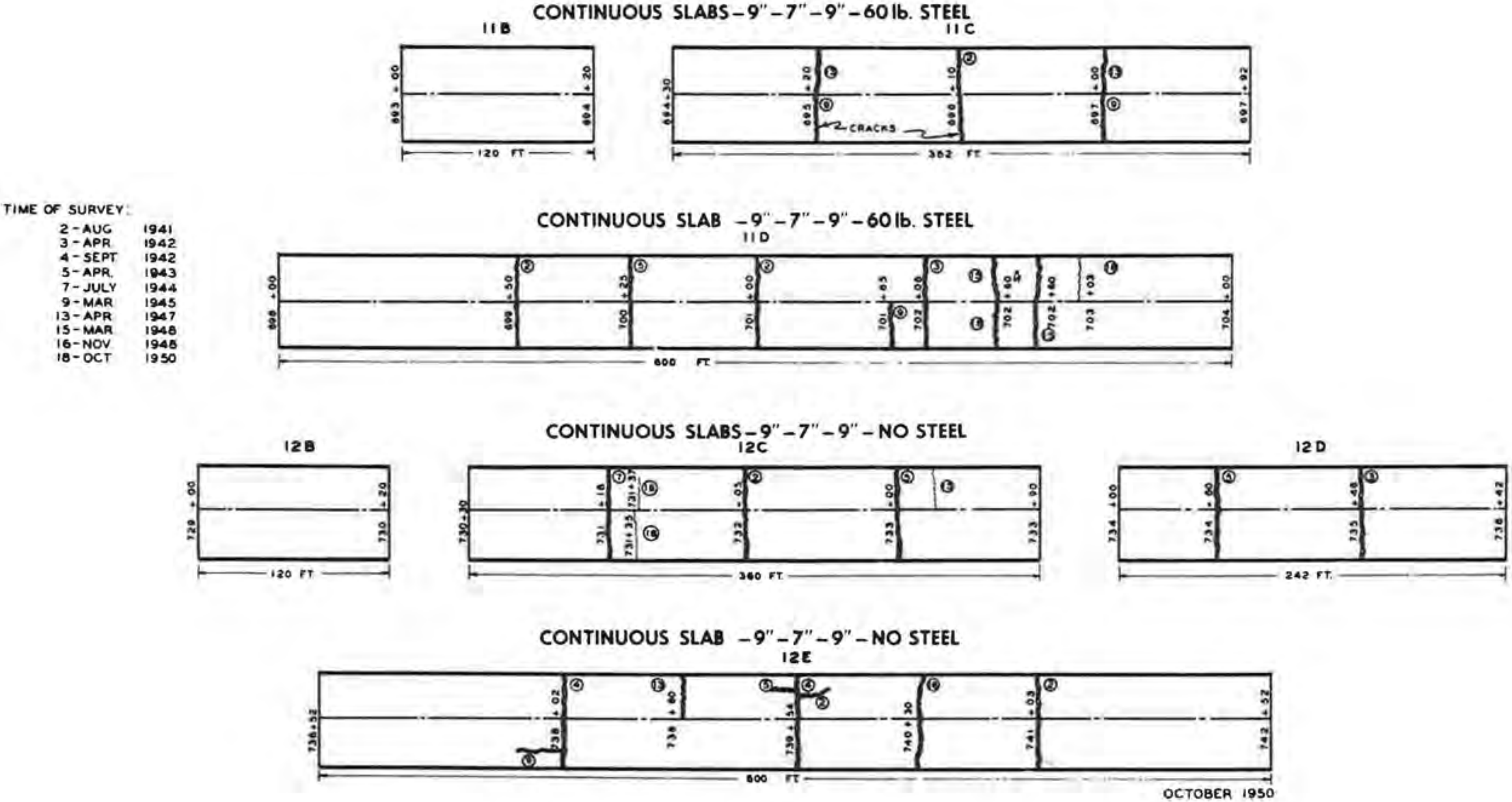


Figure 26. Condition of pavement in Series 11 and 12.

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.	
Section	Steel 11C	No Steel 12C	Steel 11D	No Steel 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

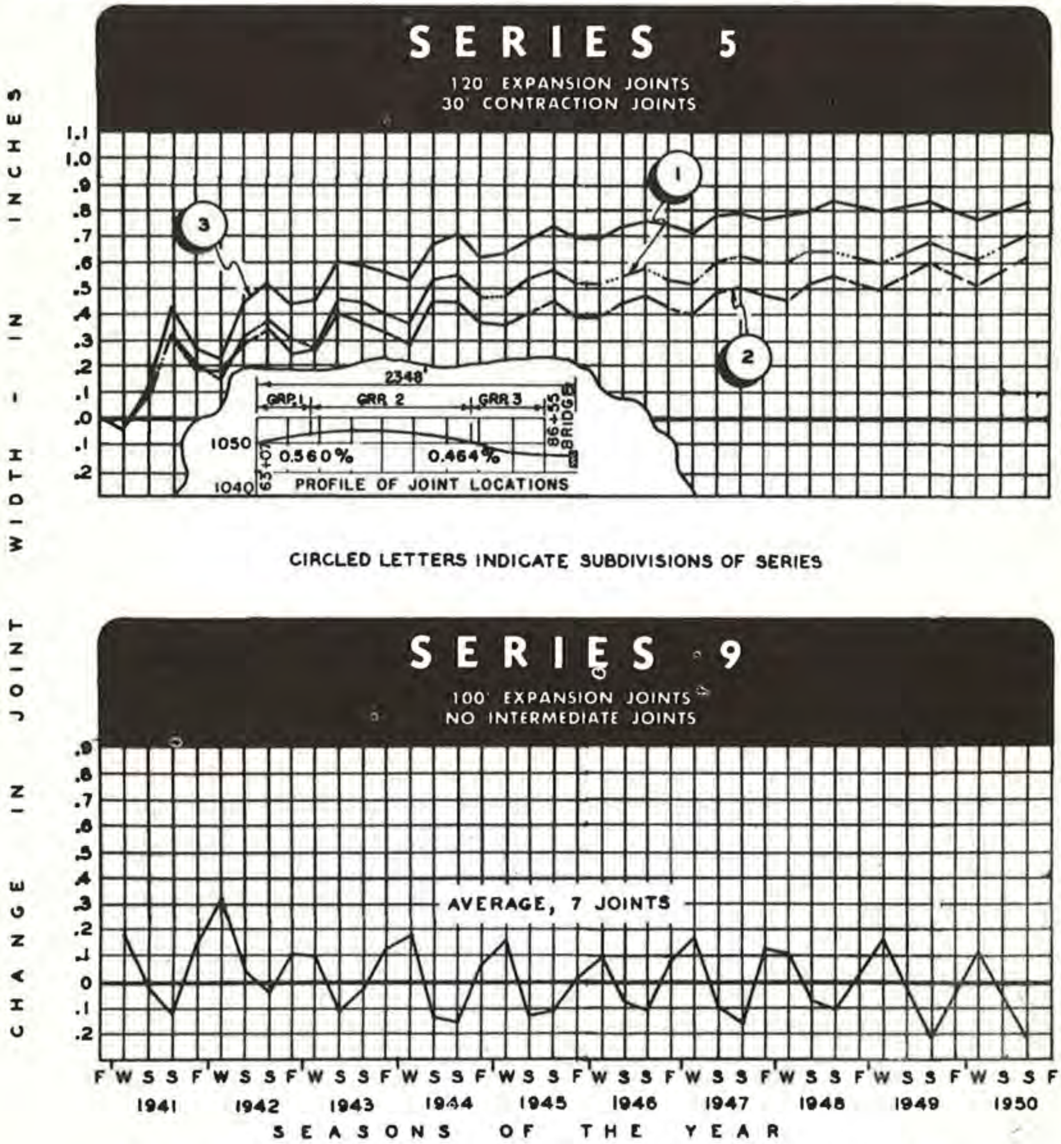


Figure 27. Seasonal changes in expansion joint width. Air chamber expansion joint.

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

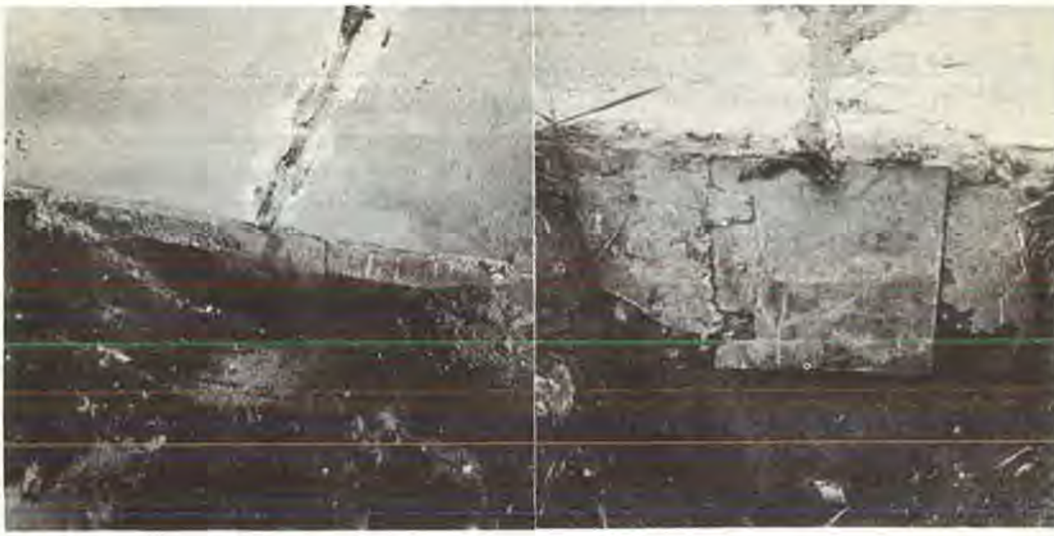


Figure 28. Air chamber expansion joint, Type DB-1 (3) Series 5, Station 79+60. (Left) at time of construction, 1940. (Right) 10 years later, 1950.

constructed somewhat differently in different locations of the Design Project, their respective performances will be discussed separately.

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.

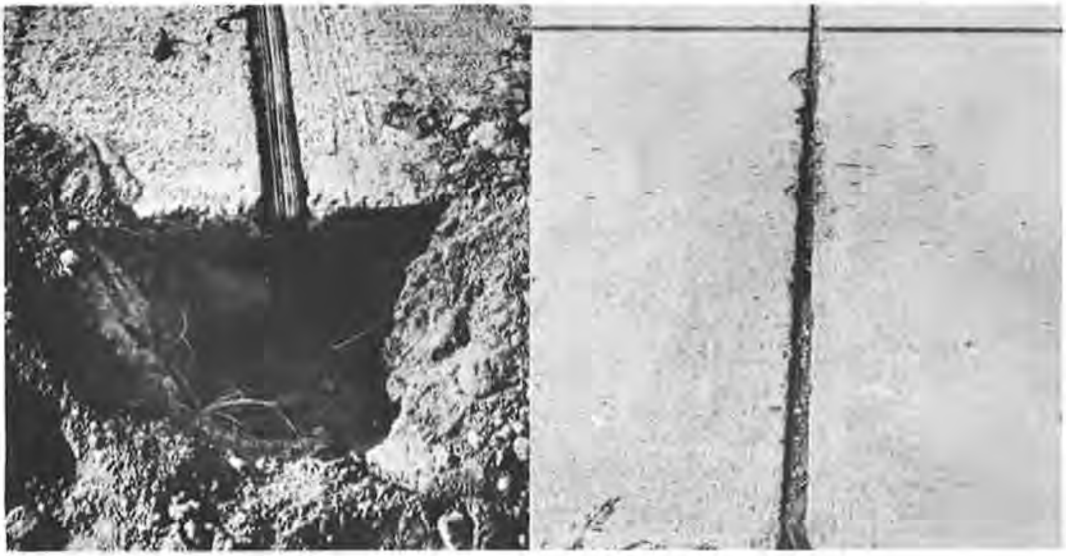


Figure 29. Air chamber expansion joint, Type TB-(4) Series 9A, Station 171+10. (Left) at time of construction, 1940. (Right) same joint 2 years later (1942) when joint had to be repaired.

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 prevention 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]		
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	Deflection Data, Loaded Slab Maximum, inches	Deflection Data, Loaded Slab Minimum, inches	Deflection Data, Loaded Slab Average, inches	(a) Average Unloaded Slab Defl. inches	(m) 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

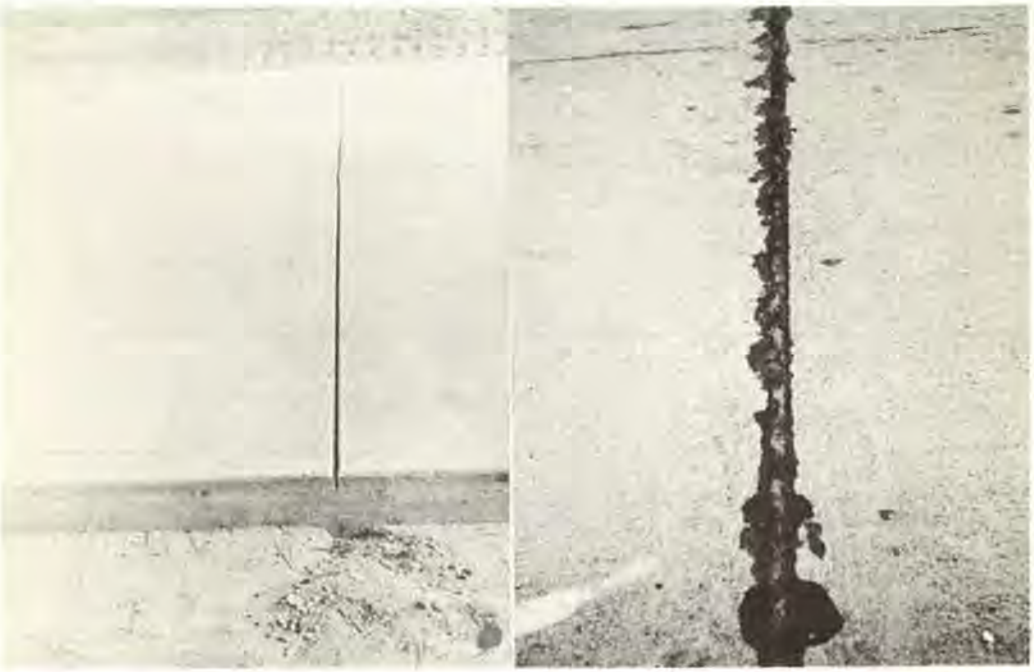


Figure 30. Type 1B contraction joint with groove, Station 64+90. (Left) joint when installed, 1940. (Right) same, joint, 1950.

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

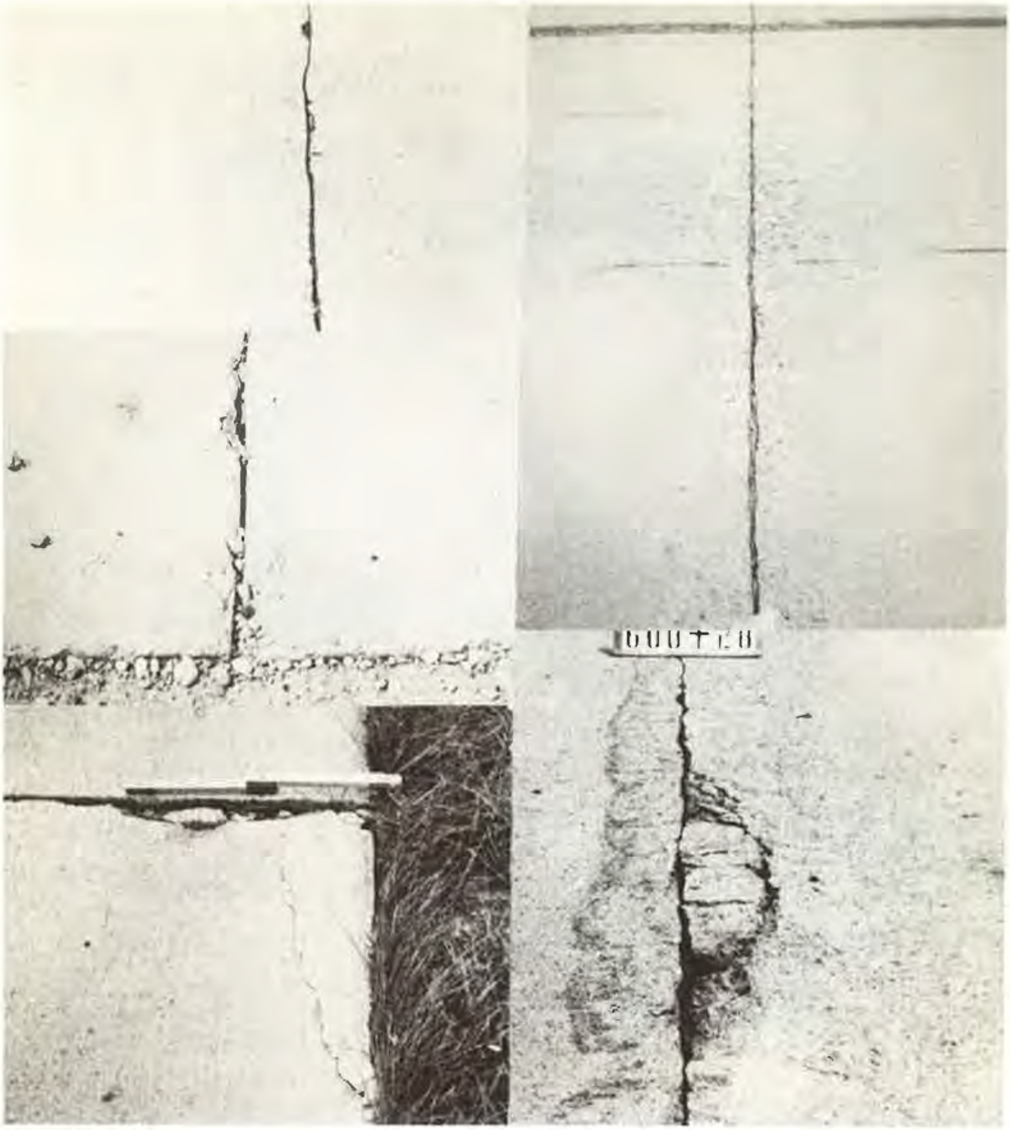


Figure 31. Typical failures common to contraction joints constructed with premolded fiber strips.

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-

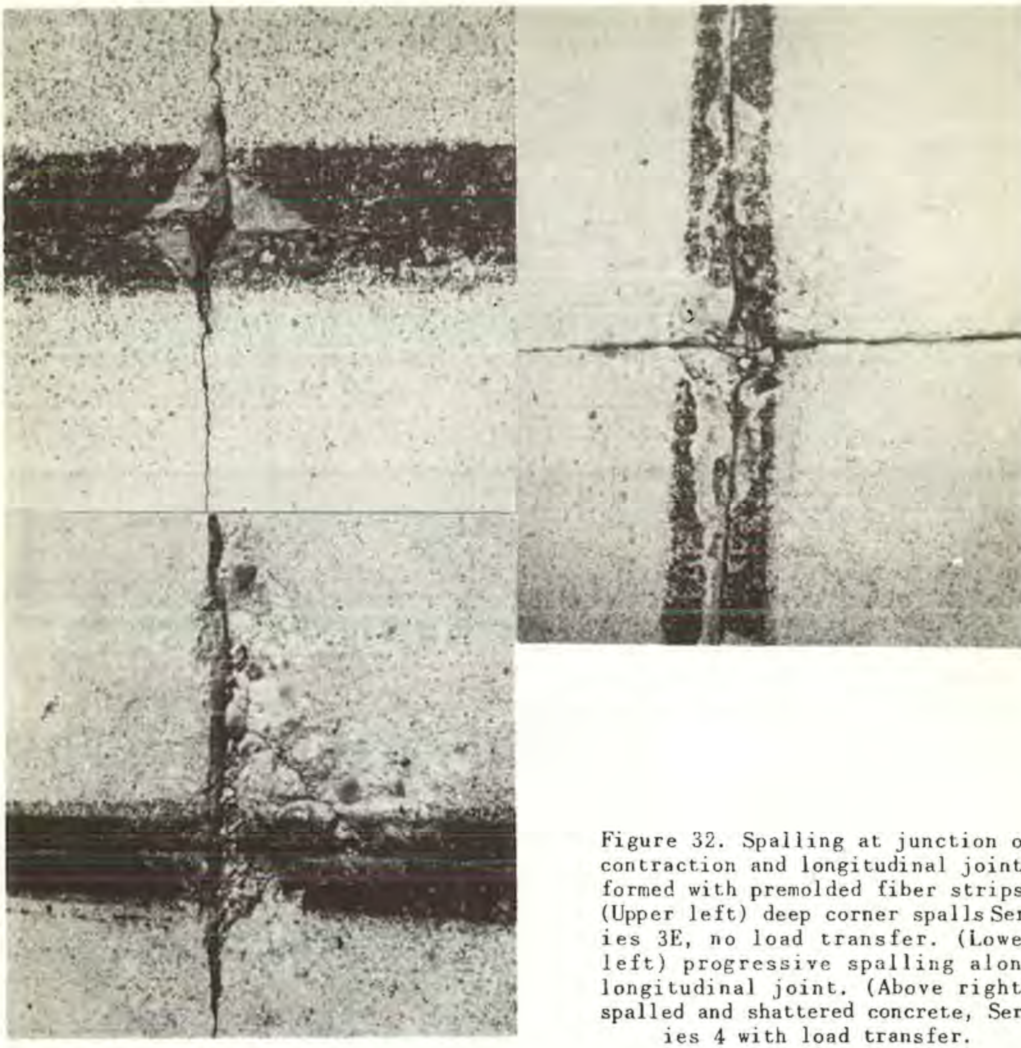


Figure 32. Spalling at junction of contraction and longitudinal joints formed with premolded fiber strips. (Upper left) deep corner spalls Series 3E, no load transfer. (Lower left) progressive spalling along longitudinal joint. (Above right) spalled and shattered concrete, Series 4 with load transfer.

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 $\frac{3}{4}$-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.

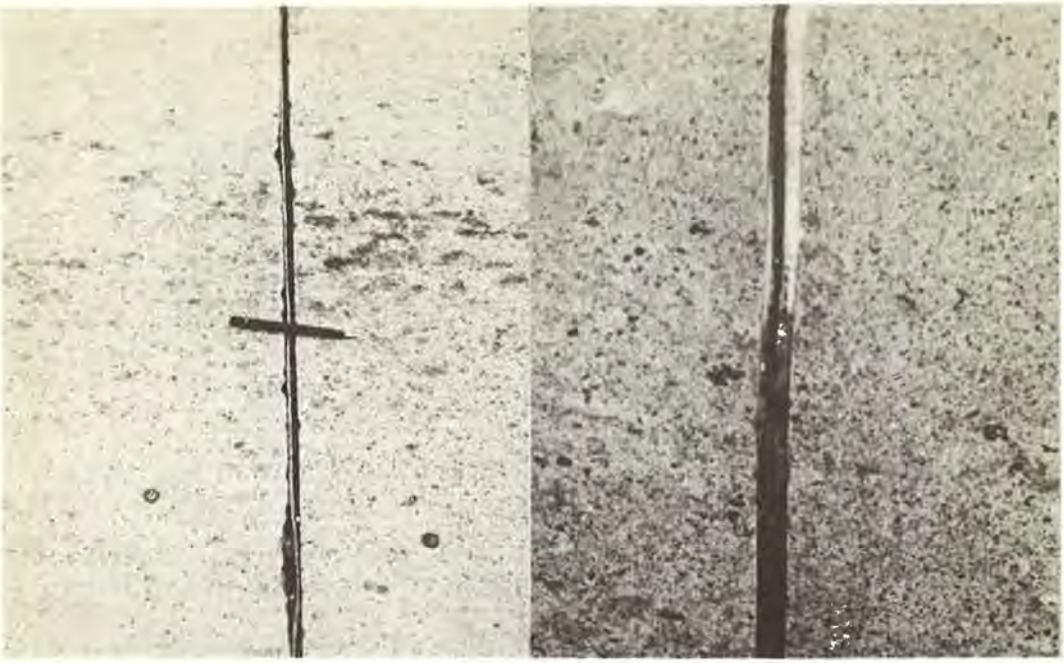


Figure 33. Typical condition of contraction joints in winter with bituminous premolded fiber strip, Series 1A, 60-ft. contraction joint spacing. Winter 1944-45. (Left) open to permit intrusion of inert material. (Right) snow and ice forms in joint.

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]		
Series	Pavement Thickness, inches	Expansion Joint Spacing	Contraction Joint Type	Load Transfer	No. of Joints	Avg. Joint Opening, inches	Deflection Data, Loaded Slab: Maximum, inches	Minimum, inches	Average, inches	(a) Average Unloaded Slab Defl. inches	(m) 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	9-6-8	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.

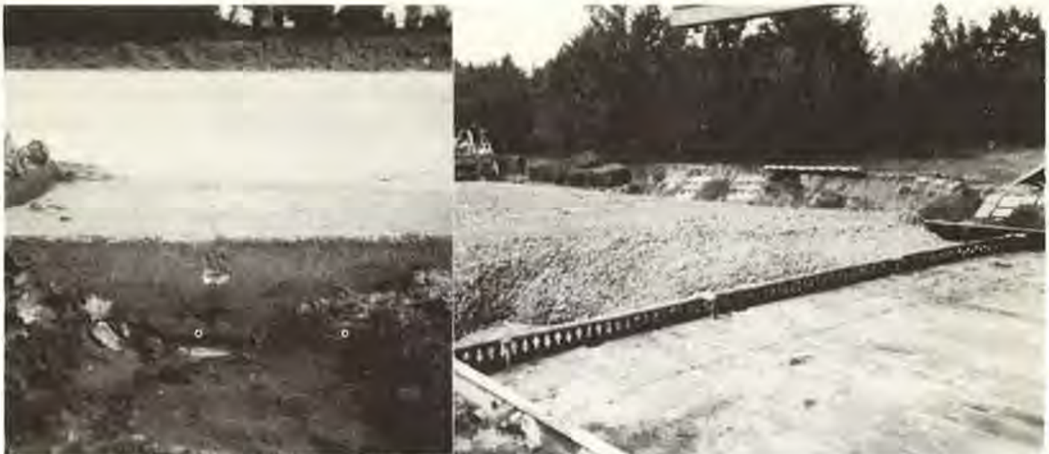


Figure 34. Type 5, Keylode contraction joint assembly. In Series 9TS. (Upper left) typical condition at time of construction, 1940, Station 181+60. (Lower left) typical condition of all similar joints after 10 years, Station 181+60. (Above right) general view of joint assembly.

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.

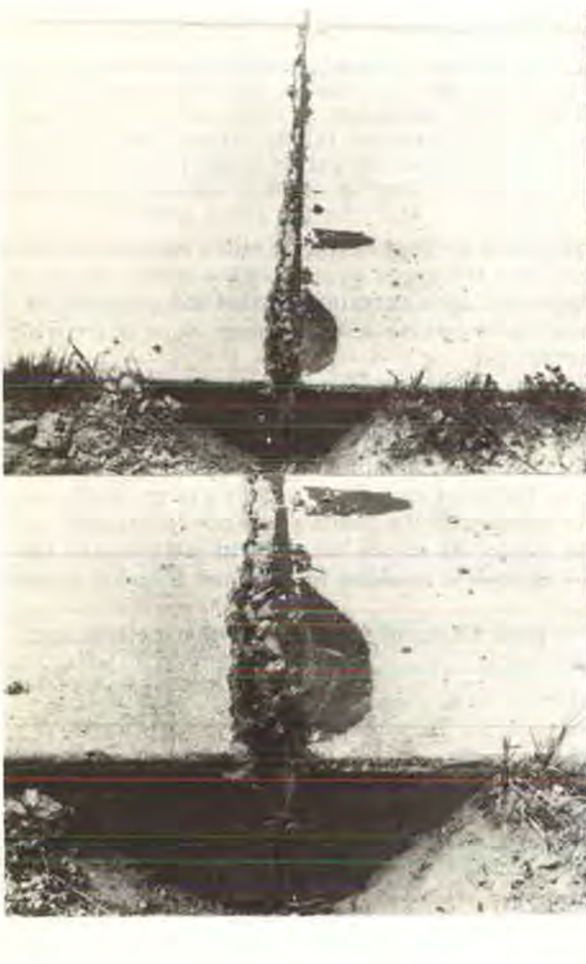


Figure 35. Type 4. Continuous plate dowel contraction joint assembly in Series 5. (Upper left) typical spalling at joint edge. (1946) Station 83+20. (Lower left) condition of joint at end, 1946. (Above right) general view of joint assembly.

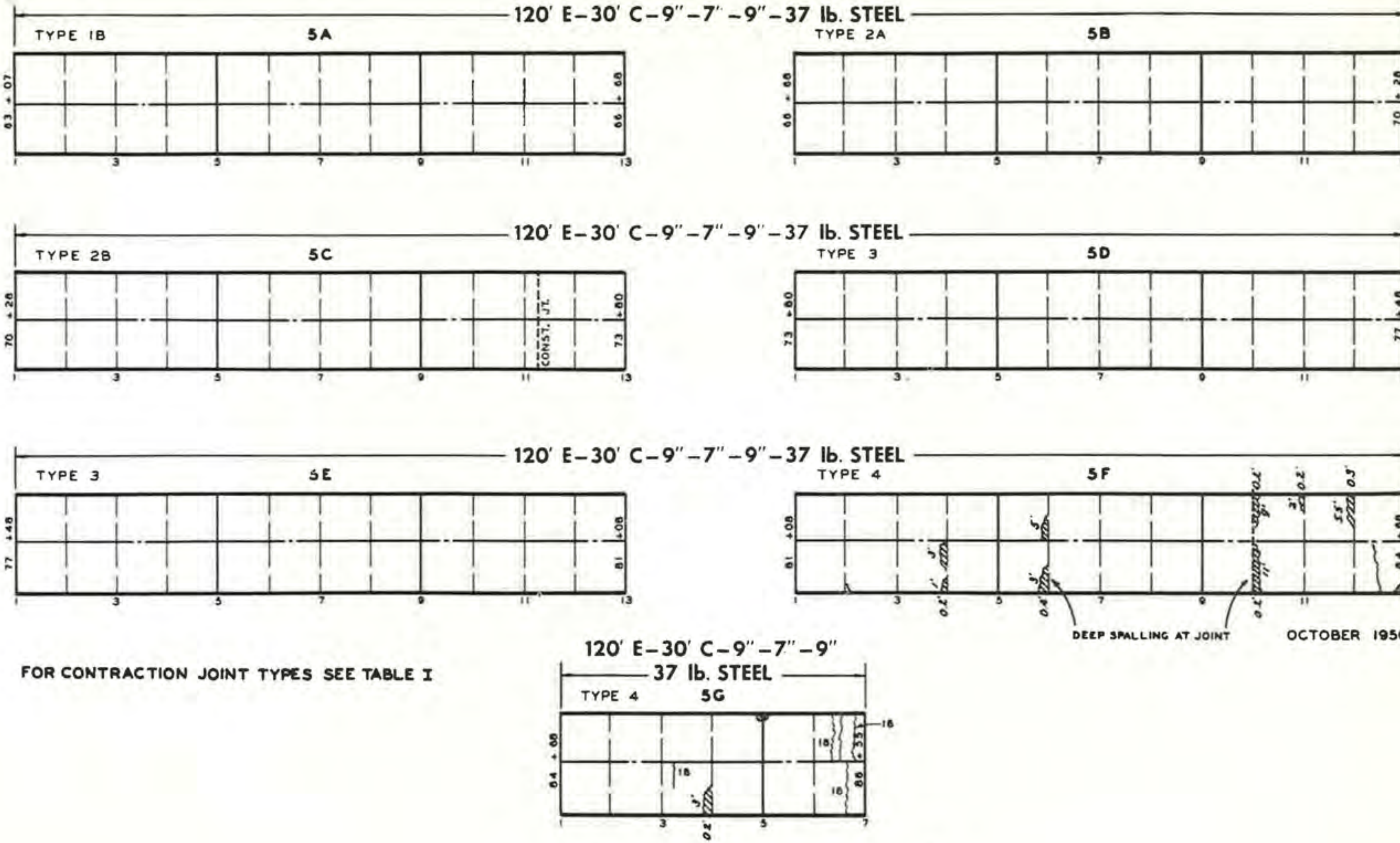


Figure 36. Condition of pavement in Series 5.

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 con-

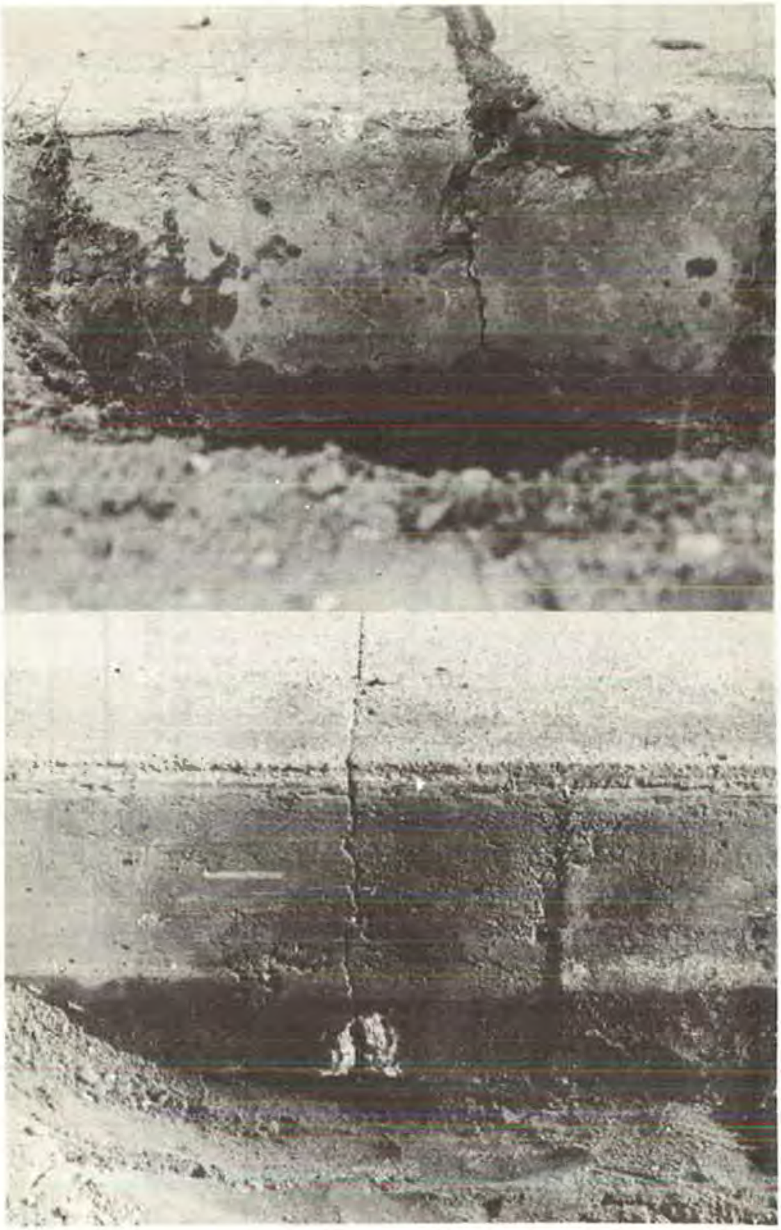


Figure 37. Effect of bottom parting strip on joint cracking. (Top) typical cracking at plane of weakness joint without parting strip at bottom. (Bottom) typical cracking when parting strip is properly installed.

sisted 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.

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).

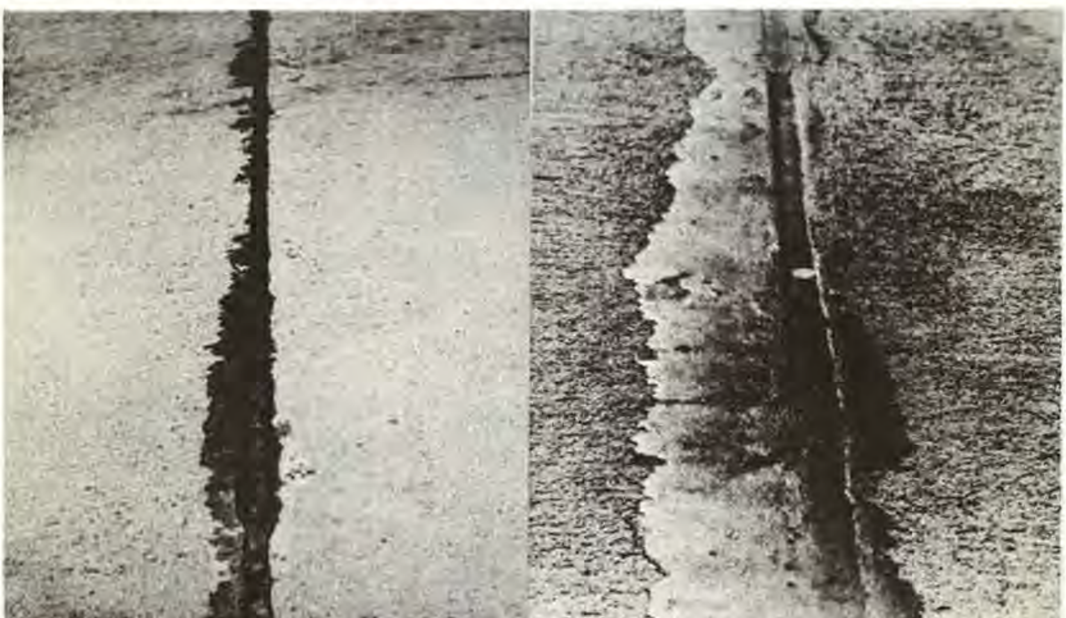


Figure 38. Present condition of bituminous-rubber material after 10 years. (Left) asphalt-latex sealing compound. (Right) thermoplastic sealing compound.

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 $\frac{3}{4}$-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

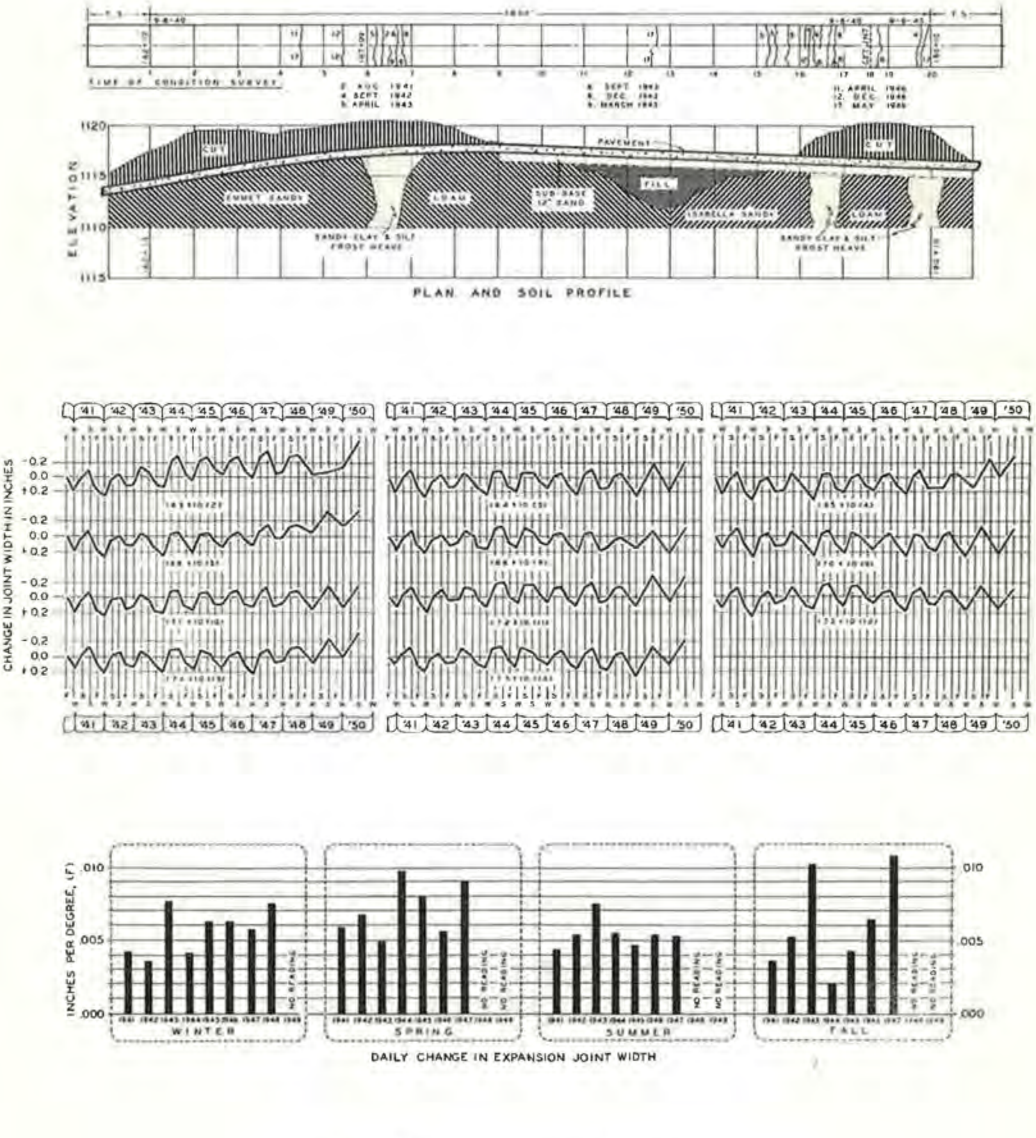


Figure 40. Stress cured concrete.

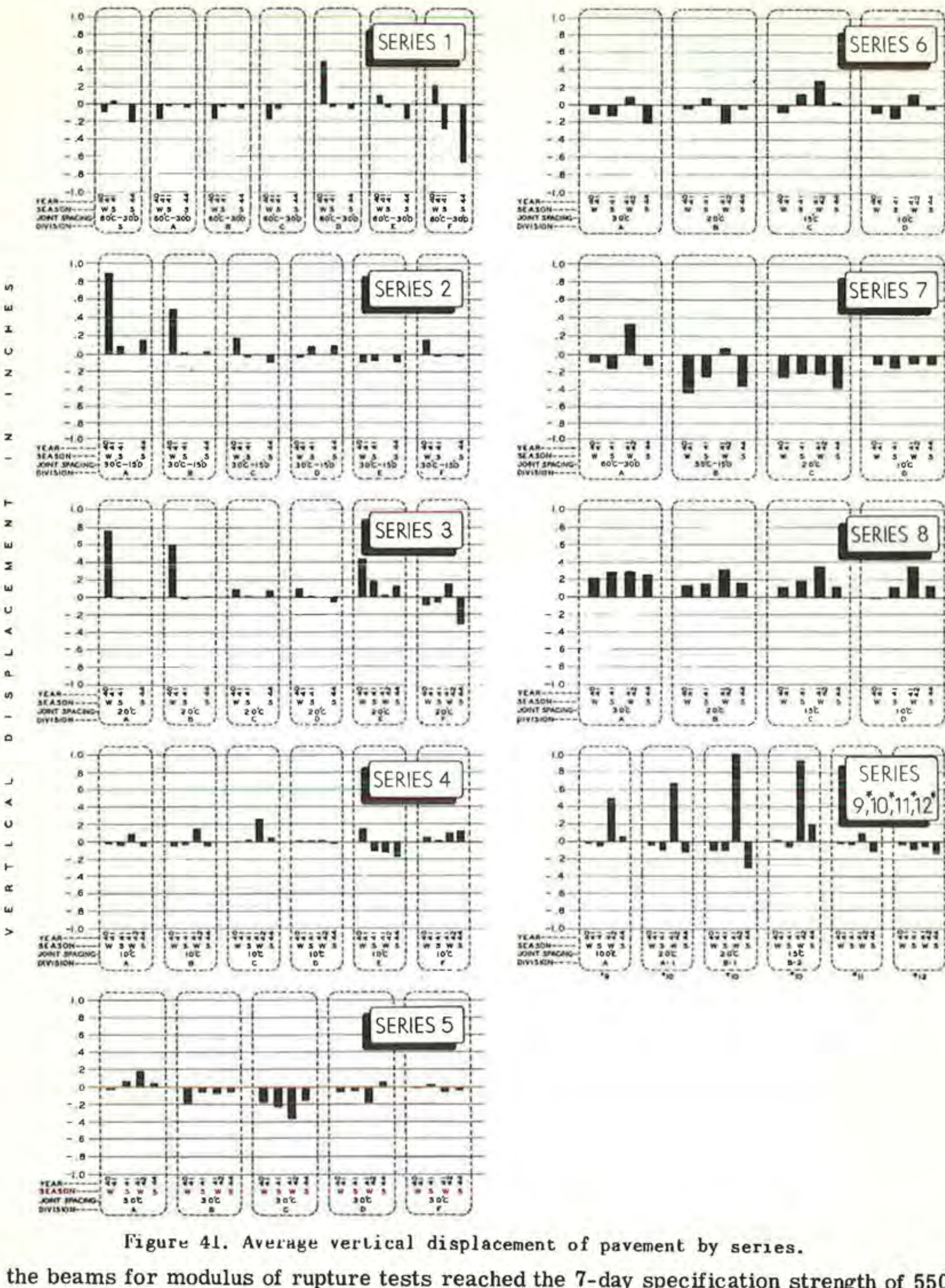


Figure 41. Average vertical displacement of pavement by series.

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.

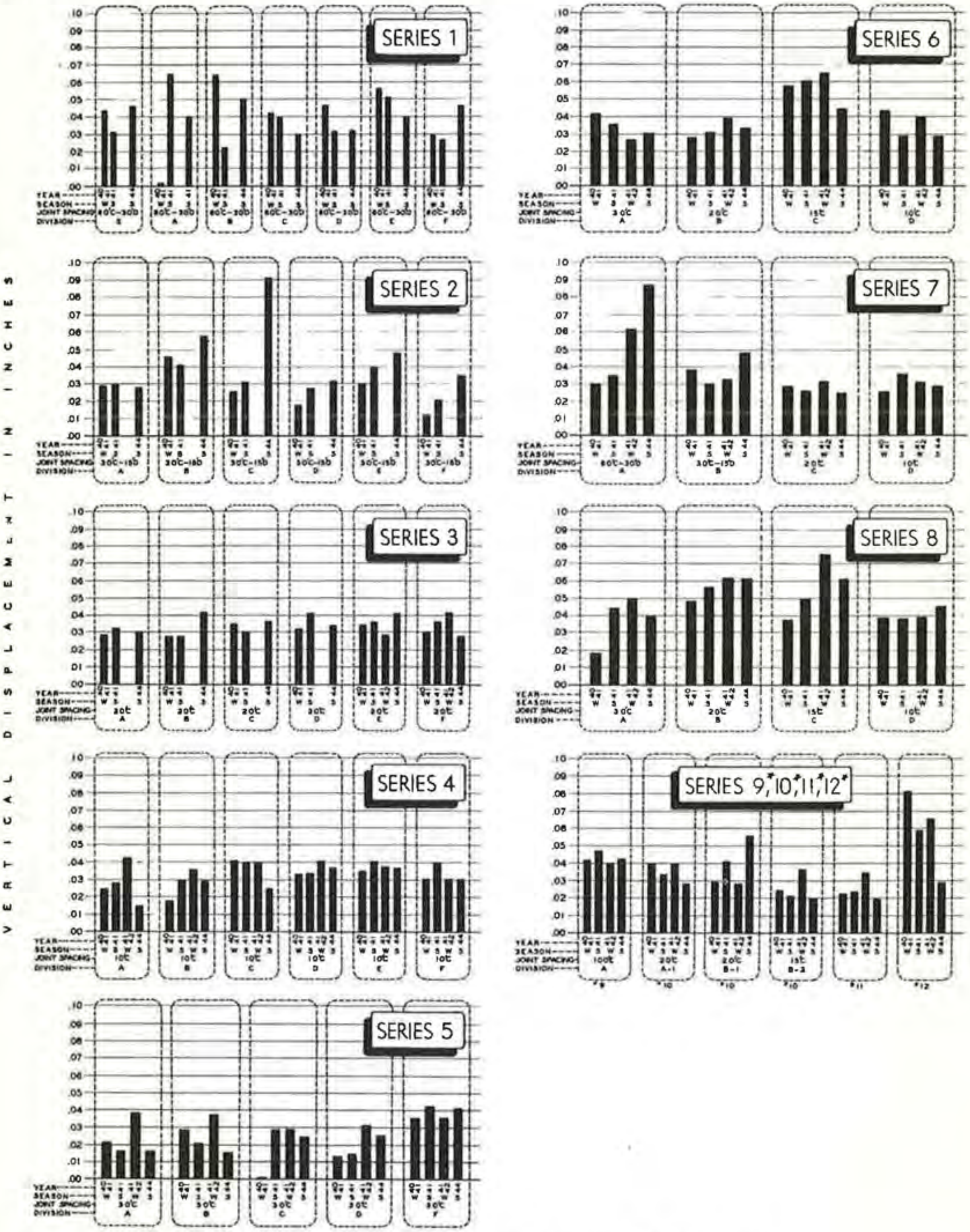


Figure 42. Average relative displacement of slab ends.

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 ⅓ 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 $\frac{1}{16}$ 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

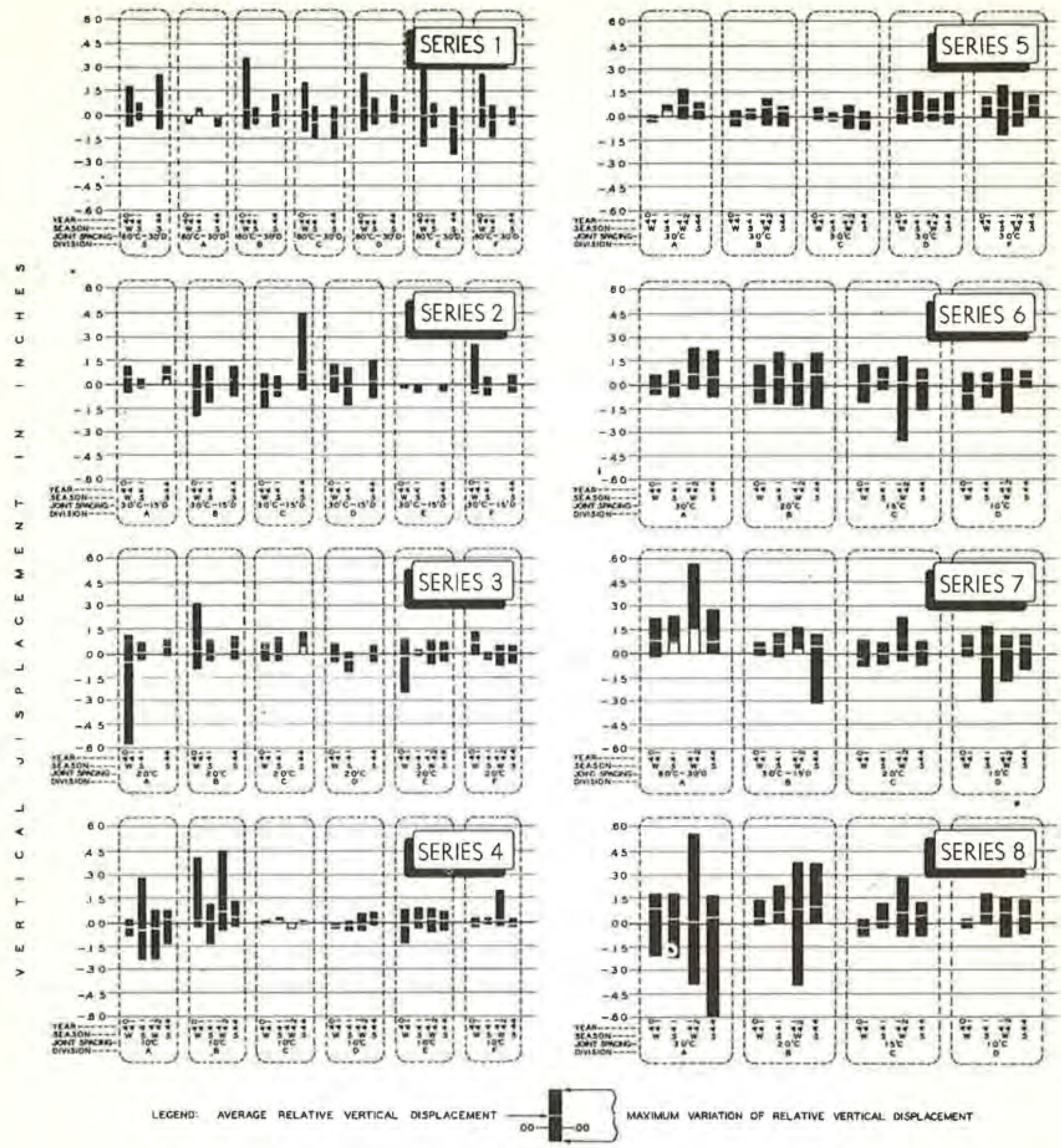


Figure 43. Relative vertical displacement of slab ends with respect to center.

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.

Report on Experimental Project in Minnesota

E. C. CARSBERG, Concrete Engineer and
P. G. VELZ, Laboratory Chief,
Minnesota Department of Highways, Division of Materials and Research

In 1940, the Minnesota Department of Highways constructed an investigational concrete pavement under regular contract and construction procedures. This project was one of a group of six built in cooperation with the Bureau of Public Roads by the States of California, Kentucky, Michigan, Minnesota, Missouri and Oregon. The purpose of these experimental projects was to study and evaluate various fundamental principles of concrete pavement design and the relative performance of such pavements over a period of years.

The Minnesota project, consisting of 8.1 miles of 22-foot concrete pavement, was constructed on State Highway 60 between Worthington and Brewster during the period from August 6 to September 20, 1940. The general layout and special design features were described in the Proceedings of the Twentieth Annual Meeting of the Highway Research Board (1940). (1)

An evaluation of the project was made in 1944 covering pertinent construction details together with data on observations and measurements taken up to, and including those of July 1944. These findings were published in Highway Research Board Research Report No. 3B (1945). (2)

The present report includes the data obtained on the project up to and including 1950.

SUBGRADE

THE subgrade is a relatively uniform, clay-loam soil with a general classification of A7-6 according to AASHO Designation M 145-49. At the time of construction, field density tests showed an average density of the upper 18 inches of 96.3 percent as compared with standard laboratory moisture-density tests. There was considerable variation in the subgrade density with a maximum of 122.5 percent and a minimum of 71.3 percent.

To date the subgrade has performed reasonably well. There has been some differential heaving at transitions from cut to fill sections. The development of high joints has been moderate, considering that experience has shown that high joints frequently occur on this type of soil. The 60-foot reinforced panels have been the only ones that have developed objectionably high joints. Pumping has not been noted on this project. Some faulting has occurred during the last few years.

CONCRETE

The aggregates used in the concrete were washed sand and gravel from a deposit located approximately 33 miles west of the project. These materials were shipped by rail and batched from a track-side proportioning plant. The properties of the aggregates are shown in Table 1.

The cement was a standard Type I cement, the properties of which are shown in Table 2.

The mix proportions, by absolute volumes, of the concrete for aggregates from Pit No. 1 varied from 1:2.788:6.442 to 1:2.834:6.385 and for Pit No. 2 the proportions were 1:2.943:6.111. The water-cement ratio varied from 5.81 to 6.11 gallons per sack of cement. The consistency of the concrete was maintained within a range of ½ to 1½ inch of slump. The concrete was placed by vibratory equipment of the tubular, internal type which operated at a frequency of 4,500 to 5,000 impulses per minute. This unit was effective in producing a high degree of consolidation. Tables 3 and 4 show the properties of the concrete as well as the results of the various strength tests.

TABLE 1

COARSE AGGREGATE PROPERTIES & TESTS

ITEM		PIT Nº 1 MAX.	PIT Nº 1 MIN.	PIT Nº 1 AV.	PIT Nº 2 MAX.	PIT Nº 2 MIN.	PIT Nº 2 AV.
Gradation	% Passing 2½" Sieve (Sq.)	100	100	100	100	100	100
	% Passing 2" Sieve "	100	100	100	100	100	100
	% Passing 1½" Sieve "	100	100	100	100	100	100
	% Passing ¾" Sieve "	85	50	65	77	53	64
	% Passing ⅜" Sieve "	38	22	29	38	20	27
	% Passing No. 4 Sieve "	5	1	3	5	1	3
	Fineness Modulus	7.27	6.72	7.03	7.26	6.80	7.06
L.A.R. Loss	"A" Gradation – Total Sample	36%	32%	35%	36%	30%	33%
	Poorest 15% Fraction	56%	42%	48%	53%	43%	47%
Decant. Loss	At Producing Plant	0.58%	0.15%	0.36%	0.42%	0.27%	0.35%
	At Batching Plant	1.07%	0.34%	0.81%	1.00%	0.60%	0.82%
Lithological Analysis	% Hard Rock Particles			41.62			40.91
	% Limestone Particles			58.01			58.72
	% Sandstone Particles			.00			.00
	% Schist and Disintegrated Particles			.06			.02
	% Shale			.10			.09
	% Spall Material			.21			.26
	% Coated Particles			.00			12.95
	% Crushed Particles			10.99			12.91
% Voids – Dry and Rodded				32.88			34.48
Specific Gravity				2.63			2.64
% Absorption				1.95			1.95

FINE AGGREGATE PROPERTIES & TESTS

ITEM		PIT Nº 1						PIT Nº 2					
		LAB. TESTS			FIELD TESTS			LAB. TESTS			FIELD TESTS		
		MAX.	MIN.	AV.	MAX.	MIN.	AV.	MAX.	MIN.	AV.	MAX.	MIN.	AV.
Gradation	% Pass. ⅜" Sieve	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
	% Pass. No. 4 Sieve	99.8	99.5	99.6	—	—	—	99.8	99.7	99.8	—	—	—
	% Pass. No. 6 Sieve	99.5	97.1	99.0	100.0	86.0	99.0	99.6	99.4	99.4	100.0	100.0	100.0
	% Pass. No. 10 Sieve	92.6	91.1	92.0	94.0	86.0	89.0	93.8	88.4	92.2	96.0	86.0	92.0
	% Pass. No. 20 Sieve	66.6	65.0	65.9	70.0	52.0	63.0	66.7	61.4	64.1	70.0	54.0	64.0
	% Pass. No. 50 Sieve	11.6	10.0	11.0	13.0	6.0	10.0	15.2	9.8	13.1	15.0	6.0	11.0
	% Pass. No. 100 Sieve	3.3	1.5	2.3	—	—	—	3.2	1.7	2.4	—	—	—
	Fineness Modulus	2.73	2.61	2.66	3.18	2.56	2.81	2.81	2.57	2.65	2.96	2.48	2.68
Decantation Loss – %		1.00	0.90	0.97	0.70	0.39	0.54	1.40	0.50	0.92	0.67	0.34	0.55
Color Plate		I	I	I	I	I	I	I	I	I	I	I	I
Strength Ratio		7d. = 1.268			28 d. = 1.222			7d. = 1.137			28d. = 1.196		
% Shale		Max. = 0.36 Min. = 0.12 Av. = 0.20						1 Test = 0.20					
Specific Gravity							2.64						2.62
% Absorption							0.66						0.85

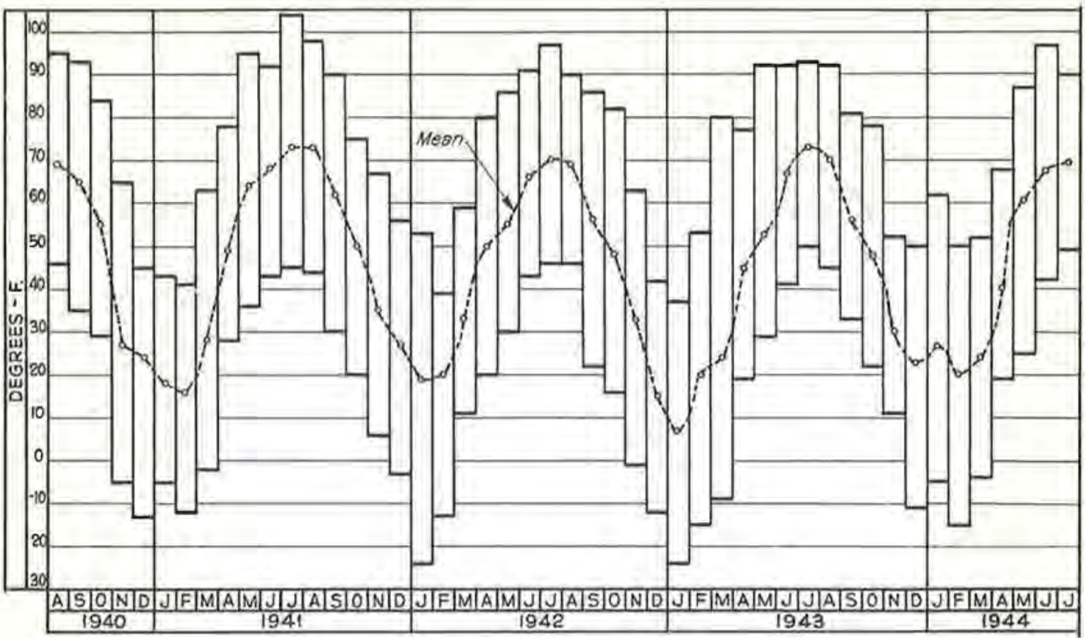


Figure 1. Air temperature data.

TRAFFIC

The amount of traffic on this project has increased considerably since 1940. There has also been an increase in the number of the heavy truck and trailer units. Table 5 shows the 24-hour average annual daily traffic for various years from 1936 through 1952.

CLIMATOLOGICAL DATA

The monthly maximum, minimum and mean temperatures from August, 1940 through July, 1944 are shown in Figure 1. Figure 2 shows the monthly precipitation for the same period. These data may be considered typical for the project area for the years subsequent to 1944.

DAILY CHANGES IN JOINT OPENINGS

The daily variations in joint openings were determined at 18 expansion joints and 52 contraction joints. The measurements were made at 3-hour intervals throughout a 24-hour period during each season from October, 1940 through July, 1942. Thereafter, readings were made in winter and summer until July of 1944. Only the data obtained on July 24, 1944 are presented in detail; because, on this date, the daily range in temperature was greater than at any previous period when these measurements were made. The joints were selected so as to provide data on panel lengths of 15, 20, 25, 30 and 60 feet. Expansion joint intervals varied from 120 feet to one mile.

Figures 3 through 7 show the movement of the joints on typical sections with 120-foot expansion joint intervals where the panel lengths were 25, 30 and 60 feet. Unfortunately no provision was made to obtain measurements on 120-foot sections containing 15 and 20-foot panels. These figures show the extremely large movements that were associated with the 60-foot reinforced panels as compared with those of the 25 and 30-foot panels. The movements at the expansion joints in the 60-foot design were about twice the movement of the intermediate contraction joint and about four times the movement of the expansion joints associated with the 25 and 30-foot panels. All of the 60-foot panels showed restraint or closing of the contraction joint at a point equivalent to about one-half the daily rise in temperature, after which the entire 120 feet continued to expand as a unit, thus accounting for the large movements at the expansion

TABLE 2

CEMENT PROPERTIES & TESTS

TESTS ON COMPOSITE SAMPLE

	NORMAL	INITIAL SET	FINAL SET	7 DAY TENSILE	28 DAY TENSILE	SPEC. GRAVITY	SPEC. SURF. AREA	AUTOCLAVE
PHYSICAL TESTS	23.6	2:40	5:10	377	428	3.131	1573	0.05%

		7 DAY	14 DAY	28 DAY	180 DAY
MORTAR CUBE TESTS	Using Graded Ottawa Sand. (A.S.T.M. - C 109 - 34 T.)	1718	2258	3068	3506

			7 DAY	14 DAY	28 DAY	90 DAY	180 DAY
SPECIAL MORTAR TESTS	Using a local commercial sand from Shiely St. Paul Pit. Mix = 1:2.655 by Abs. Vol. Flow = 200 ± 05. Moist air cure.	FLEXURAL TESTS	900	1188	1259	1242	1261
		COMPRESSION TESTS	4418	5345	7385	7743	8331

CHEMICAL ANALYSIS									
ANALYSIS BY WT. - %									
	Ig. Loss	CaO	MgO	Fe_2O_3	Al_2O_3	SO_3	SiO_2	Free CaO	TOTAL
	0.93	64.4	1.85	2.94	5.91	1.47	21.35	0.93	99.78
CALCULATED CHEMICAL COMPOSITION - % BY WT.									
	C_2S	C_3S	C_3A	C_4AF	$CaSO_4$	Free CaO	Free MgO	Free Mn_2O_3	Ig. Loss
	25.0	48.0	10.8	8.9	2.5	0.93	1.85	—	0.93

ROUTINE TESTS ON CARS SHIPPED TO PROJECT

	NO. OF CARS	PERIOD USED	AGG'S. USED	7 DAY TENSILE	28 DAY TENSILE	INITIAL SET	FINAL SET
AV. OF ALL TESTS ON INDIVIDUAL CARS	31	8/6/40 – 8/20/40	Pit No 1	363.4	441.2	3:19	6:12
	44	8/20/40 – 9/20/40	Pit No 2	352.0	440.6	3:27	6:17

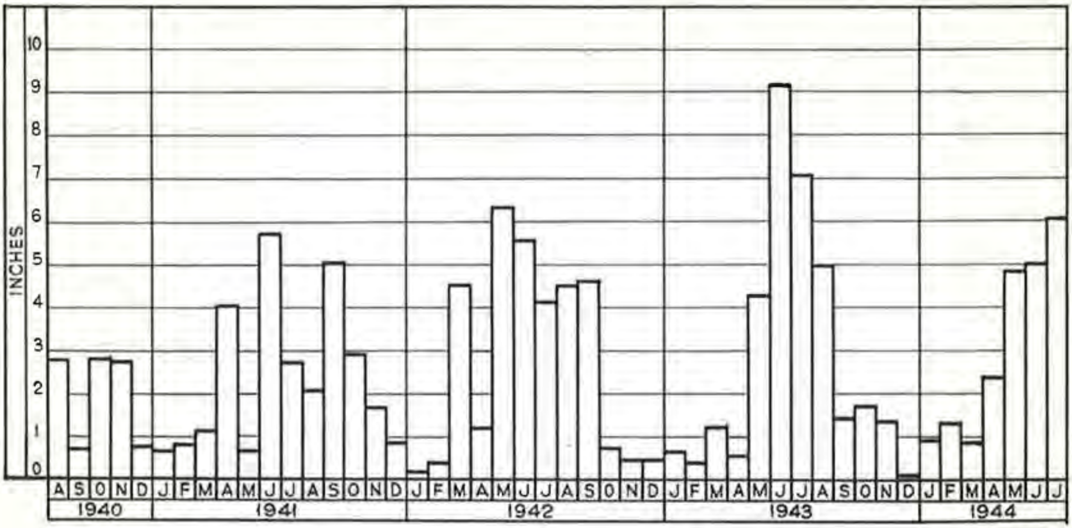


Figure 2. Precipitation data.

joints. This illustrates the difficulty which may be expected in keeping such joints sealed.

Figures 8 and 9 show the movement of joints between 15 and 30-foot panels on sections having expansion intervals of 420 feet. The movements associated with the 15-foot panels were considerably less than where 30-foot panels were used. The movement of the 30-foot joints were but slightly smaller than those of similarly spaced joints on 120-foot sections.

Figures 10 and 11 show the movement of joints spaced at 15 and 30 foot intervals on sections having approximately 800 feet between expansion joints. It is interesting to note that, in these cases, the movements of the contraction joints were of about the same magnitude for both the 15 and 30 foot spacing, particularly in the central portions of the sections. It is also of interest to note that the movement of the expansion joints on these sections was about 0.02 inch as compared to the much larger movements associated with the shorter expansion joint intervals.

Figure 12 shows the daily movement of joints over Division 9, which was about one mile in length and contained no expansion joints except at each end. The panel lengths were variable, ranging from 15 to 30 feet. There was little difference between the contraction joint movements on this division and those near the center of the 800-foot sections; also the expansion joint movements were similar. This figure provides a direct comparison of the movements associated with 30 and 15-foot panels under identical conditions of

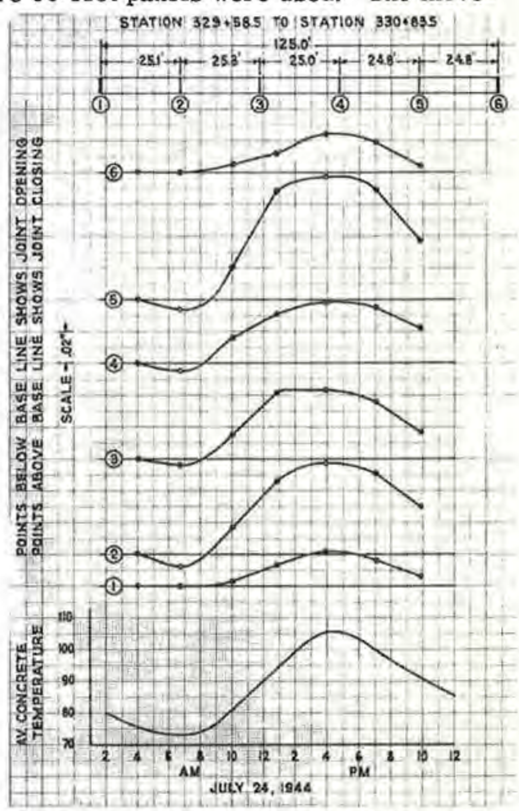


Figure 3. Daily changes in joint openings.

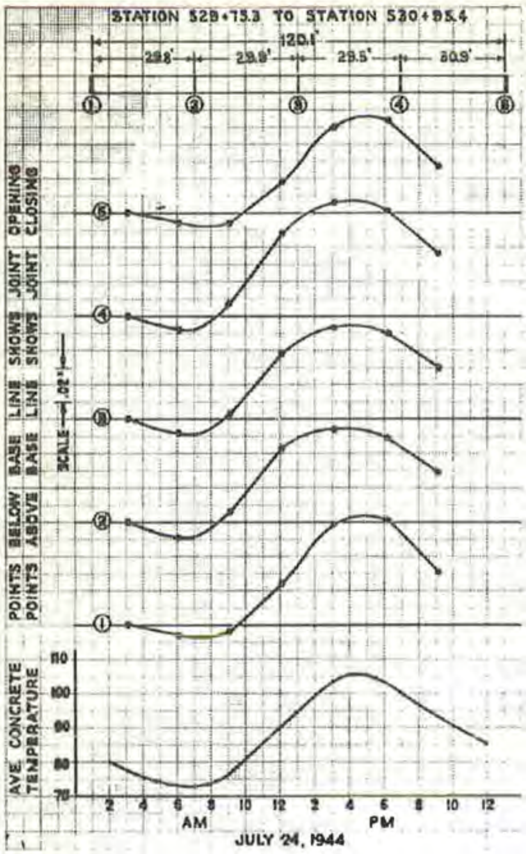


Figure 4. Daily changes in joint openings.

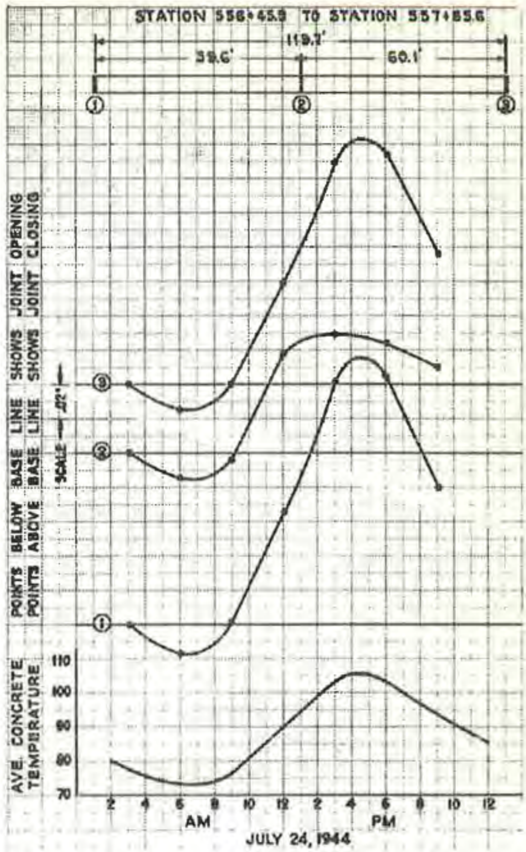


Figure 5. Daily changes in joint openings.

restraint. Even under these conditions, the joints spaced at 15 feet showed a daily range in movement of 0.027 inch while the joints spaced at 30 feet showed about 0.04 inch.

These figures indicate more or less restraint through the central portions of the longer expansion-interval sections as indicated by the flat tops of the curves over the period of highest daily temperature. There was little difference in this respect between the 800-foot and one mile sections. Wherever an expansion joint was installed there was a tendency for the contraction joints in the immediate vicinity to develop relatively large openings. This was true from the viewpoint of seasonal and permanent movements as well as the daily movements. In general, the daily changes in the contraction joint openings decreased as the expansion-joint interval increased and the contraction-joint interval decreased. Short panels and wide spacing or omission of expansion joints, therefore, would appear to be beneficial in reducing leakage of surface water through the joints and also in providing a maximum degree of load transfer across the joints.

ANNUAL AND PERMANENT CHANGES IN JOINT OPENINGS

The annual and permanent changes in joint openings were determined from measurements of 408 contraction joints and 30 expansion joints. These 438 joints were all that could be considered of the 714 joints originally provided for these measurements because of the desire to eliminate the influence of cracked panels. On this basis it was possible to obtain data on all contraction and expansion joint intervals except the 125-foot expansion - 25-foot contraction sections. Measurements were made at the following times:

1940 - October (Initial measurements two weeks after completion of project) 1941 - February, May, July and November; 1942 - February, May, July and August; 1943 -

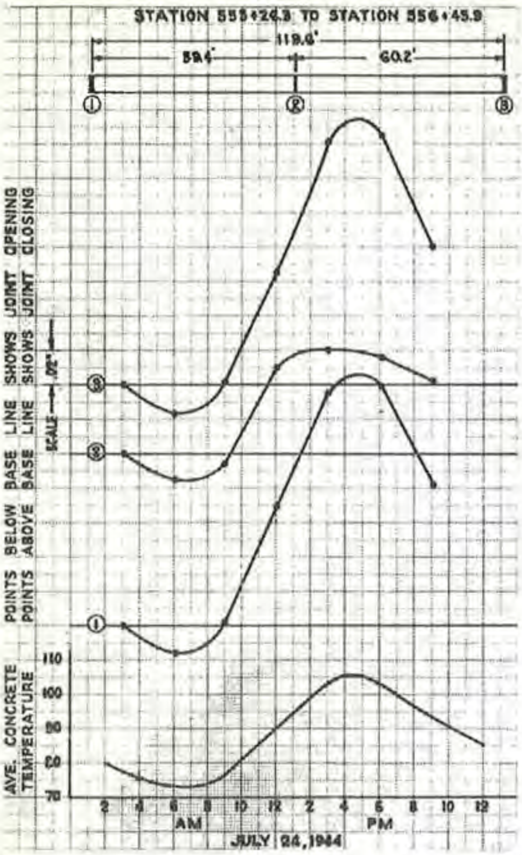


Figure 6. Daily changes in joint openings.

February and August; 1944 - January and July; 1945 - August; 1948 - February and August.

The measured changes in the joint openings are shown graphically in Figures 13 through 35. These figures, in most cases, show a composite plotting of the average values of two or more similar sections.

Figures 13 through 25 permit direct comparison of joint movements both seasonal and from year to year and show how the position of a contraction joint in a section influences its movement. On the left hand side of these figures is a horizontal decimal scale running from zero to 1. 0. This represents one-half the expansion joint interval regardless of the actual length in each case. The zero end of the scale is the expansion joint end and the value 1. 0 corresponds to the mid-point of the interval. The winter and summer readings are shown in individual blocks for each year. The departure in the opening of the contraction joints is plotted in inches as indicated by the vertical scale. On the right side of the figures is shown the closure in inches of the expansion joints from the initial opening in October, 1940.

The principal points of interest in Figures 13 through 25 are: (1) The general tendency for the contraction joint next to the expansion joint to open up considerably more than other contraction joints in the section, although the general trend indicates an increase in opening as the joint location approaches the expansion joint end of the half-section. (2) The erratic behavior of individual contraction joints during any given season or year or from year to year. (3) The proportionately large closure of the expansion joints during the first expansive cycle and the small movement of the expansion joints after eight years where the expansion interval was 400 feet or more in length.

Figure 26 shows the annual and permanent changes in joint openings for sections having 60-foot reinforced panels with alternating expansion and contraction joints. A progressive closing of the

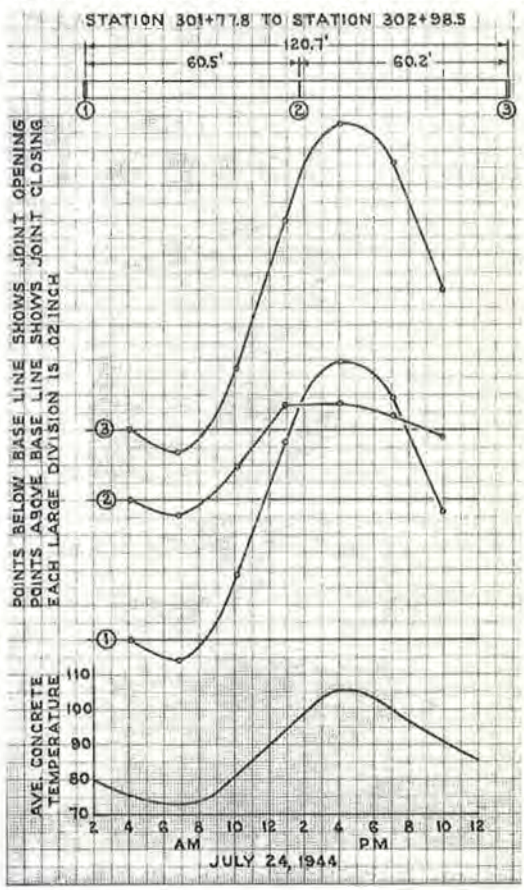


Figure 7. Daily changes in joint openings.

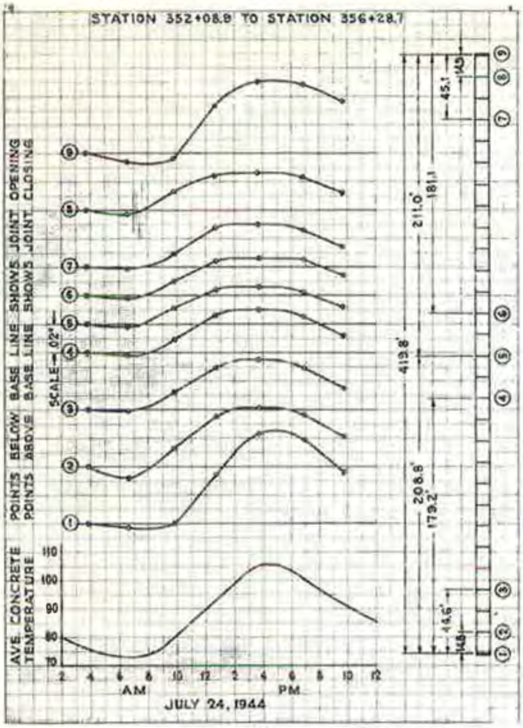


Figure 8. Daily changes in joint openings.

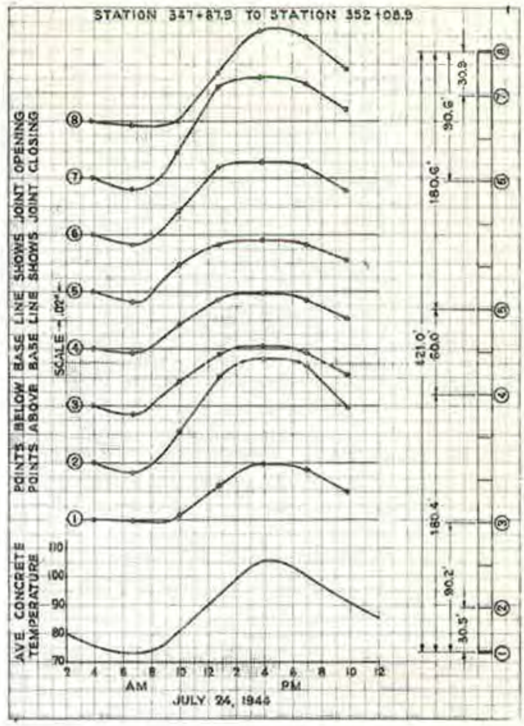


Figure 9. Daily changes in joint openings.

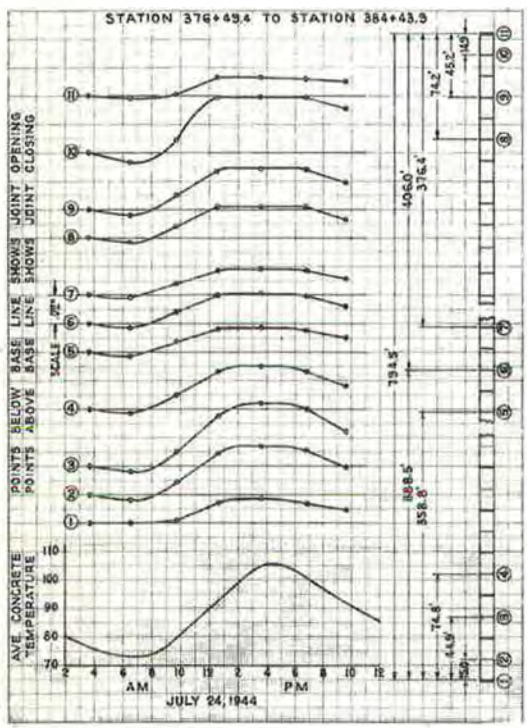


Figure 10. Daily changes in joint openings.

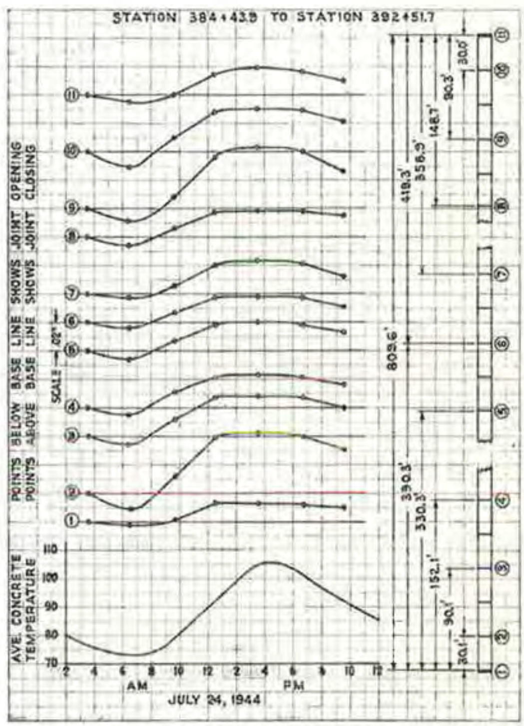


Figure 11. Daily changes in joint openings.

expansion joints and opening of the contraction joints was also indicated by these data. The magnitude of the expansion joint movements has shown no tendency to become less during the eight year period.

Due to variations in the temperature of the concrete at the time the joint measurements were read, the above figures do not clearly show the progressive permanent change in joint openings. A better comparison of the permanent change is shown in Figures 27 through 35. Here the departures from the October, 1940 openings are plotted against average concrete temperatures. Starting with the joint opening in October, 1940 as a base, the departures in joint openings from that date are plotted for periods of decreasing and increasing temperatures up to and including the summer of 1948. By projecting an ordinate from the October, 1940 point, the accumulated change in joint opening can be estimated at a common temperature.

Figures 27 through 30, showing the permanent changes in joint openings for 120-foot expansion intervals with panel lengths of 15, 20, 30 and 60 feet, indicate that, in 1940, the accumulated opening of the contraction joints per interval very nearly equalled the accumulated closure of the expansion joints.

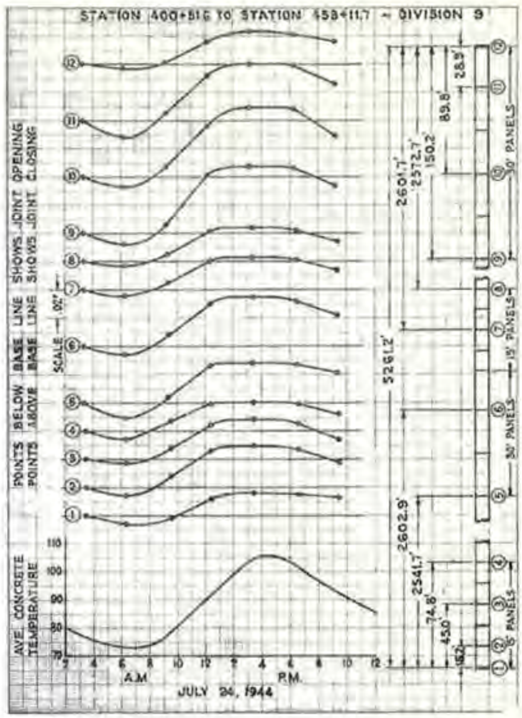


Figure 12. Daily changes in joint openings.

TABLE 3

CONCRETE STRENGTH TESTS

Flex. Tests Made in Field (**)

STA. - STA.	Pit No. 1, 7 Day: No Tests	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)	Pit No. 1, 14 Day: No Tests	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)	Pit No. 2, 7 Day: No Tests	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)	Pit No. 2, 14 Day: No Tests	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)
222+68 - 376+64.4	29	568	54.8	9.64%	30	634	59.3	9.35%	–	–	–	–	–	–	–	–
207+00 - 222+68 376+64.4 - 634+64	–	–	–	–	–	–	–	–	57	558	49.8	8.93%	55	649	57.3	8.83%

(**) Center point loading on 18" span. 6"×6"×30" Beam Specimens. {Covered with impermeable paper first 24 hrs. then immersed in water until tested.

Laboratory Core Tests

Core Heights (Actual heights shown below are the averages of no. shown.) — Compressive Strength (Corrected for H/d.)

STA.	STA.	Pavement Section	Source of Aggregates	Age of Cores (***) When Drilled	When Tested	Center Cores: No Cores	Height (Ins) Theor.	Act.	Std. Dev. (*)	Coef. of Var. (*)	Side Cores: No Cores	Height (Ins.) Theor.	Act.	Std. Dev. (*)	Coef. of Var. (*)	Compressive: No Cores	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)
222+68 293+38.5	264+30.3 376+64.4	9-6-9	Pit No 1	122	150	9	6.00"	6.19"	0.33"	5.33%	9	6.03"	6.28"	0.29"	4.62%	15	6263	489	7.81%
264+30.3	293+38.5	7"	Pit No 1	119	150	4	7.00"	6.84"	0.11"	1.61%	4	7.00"	6.83"	0.23"	3.37%	6	6319	369	5.84%
207+00 376+64.4 592+72.6	222+68 563+63 634+64	9-6-9	Pit No 2	103	150	22	6.00"	6.06"	0.14"	2.31%	17	6.03"	6.17"	0.18"	2.98%	29	5030	765	15.21%
563+63	592+72.6	7"	Pit No 2	93	151	5	7.00"	6.95"	0.27"	3.88%	4	7.00"	7.16"	0.13"	1.82%	7	4945	436	6.82%

(*) Computed in accordance with method shown in 1933 A.S.T.M. Manual on Presentation of Data.
(***) After drilling, the cores were stored in Laboratory air until 14 days prior to testing. Tested wet after 14 days in water.

TABLE 4

MISCELLANEOUS CONCRETE PROPERTIES & TESTS

Thermal Coefficient (*)	0°-40° F.	40°-80° F.	80°-120° F.
	6.82×10^{-6} per degree	6.15×10^{-6} per degree	5.45×10^{-6} per degree

(*) See Bureau of Standards Technical Paper Nº 247 for method.
Thermal Coefficient Tests made after concrete was 148 days old.

Modulus of Elasticity	NO. OF CORES	STA.	SOURCE OF AGGREGATE	TEST AGE (Days)	AV. "E" VALUES Dry	AV. "E" VALUES Wet
Special Long Cores Taken From Pavement	1	251+00	Pit Nº 1	150	4,554,200	4,197,600
	1	400+00	Pit Nº 2	150	4,116,700	3,705,100
	1	605+00	Pit Nº 2	150	3,819,000	3,633,600
6"×12" Cylinders Cast on Job	5	—	Pit Nº 1	90	—	4,501,155
	5	—	Pit Nº 1	180	—	5,080,917
	8	—	Pit Nº 2	90	—	4,764,553
	8	—	Pit Nº 2	180	—	4,988,476

Special Str. Tests Made in Lab. on Beams Cast in Field

FLEXURAL TESTS on 6"×6"×36" Beams.

STA. - STA.	SOURCE OF AGG.	28 DAYS NO.	28 DAYS AV. UNIT STR.	90 DAYS NO.	90 DAYS AV. UNIT STR.	180 DAYS NO.	180 DAYS AV. UNIT STR.	1 YR. NO.	1 YR. AV. UNIT STR.	2 YR. NO.	2 YR. AV. UNIT STR.
222+68 - 376+64.4	Pit No. 1	11	631	11	699	10	732	5	809	5	841
207+00 - 222+68 376+64.4 - 634+64	Pit No. 2	19	599	19	667	19	733	9	746	8	743

COMPRESSION TESTS ON MODIFIED CUBES
(Using sections of broken beams)

STA. - STA.	AGG. SOURCE	NO.	AV. UNIT STR.	NO.	AV. UNIT STR.	NO.	AV. UNIT STR.	NO.	AV. UNIT STR.	NO.	AV. UNIT STR.
222+68 - 376+64.4	Pit No. 1	11	4243	11	5142	11	4738	10	5564	10	4219
207+00 - 222+68 376+64.4 - 634+64	Pit No. 2	19	4829	20	5252	20	5166	22	5341	17	5835

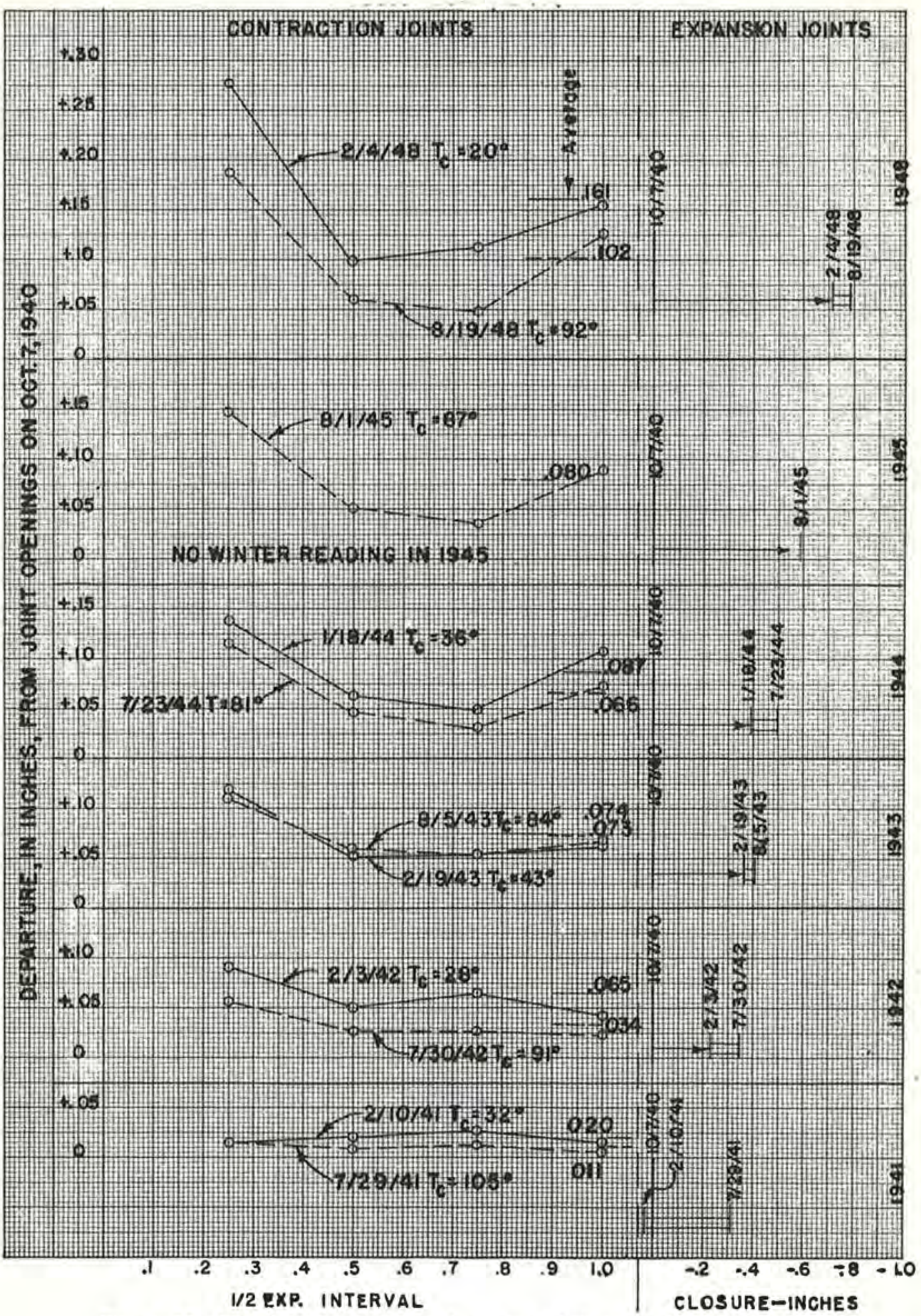


Figure 13. Annual and permanent changes in joint openings, expansion joint interval=120 feet, contraction joint interval=15 feet.

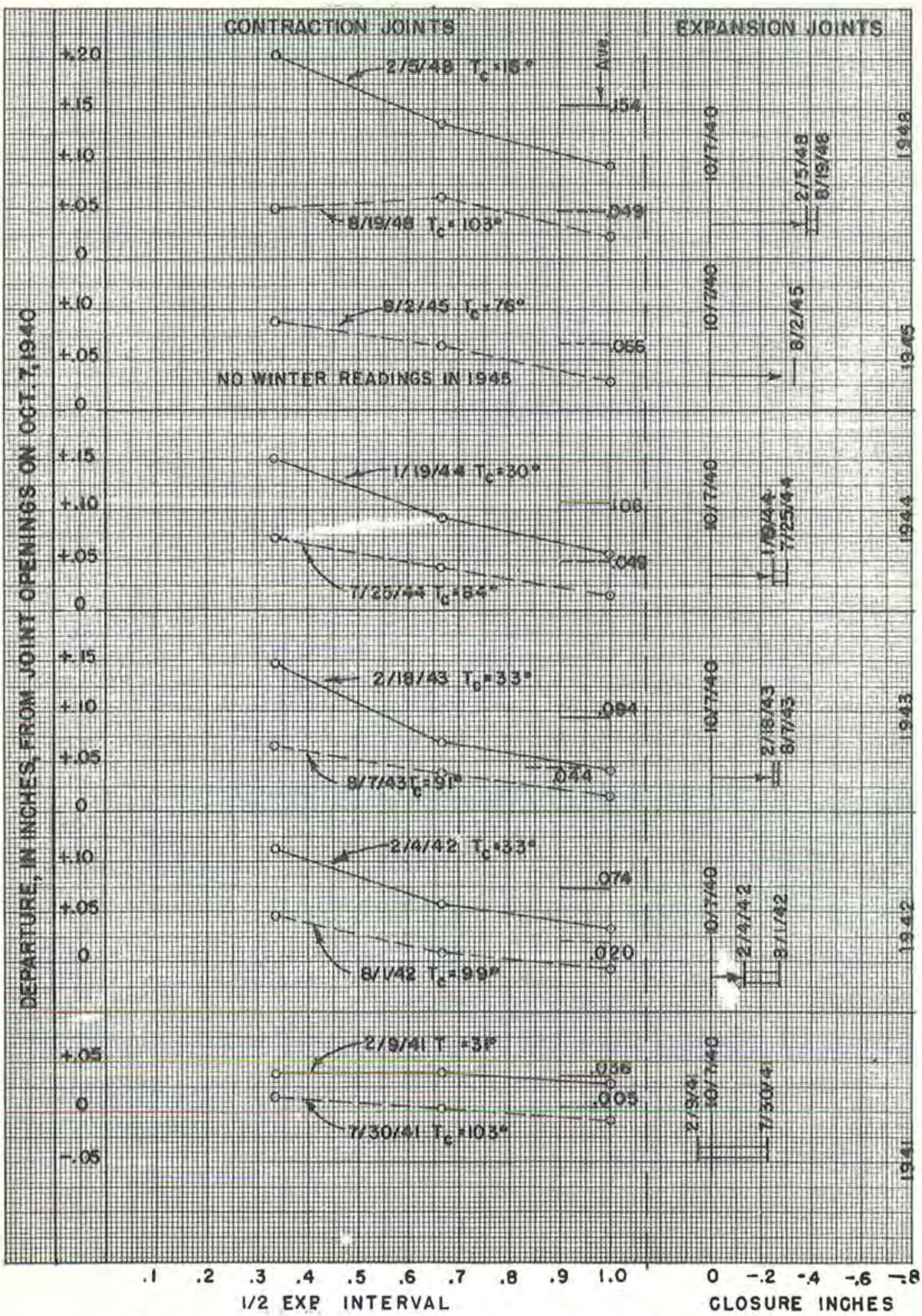


Figure 14. Annual and permanent changes in joint openings, expansion joint interval =120 feet, contraction joint interval=20 feet.

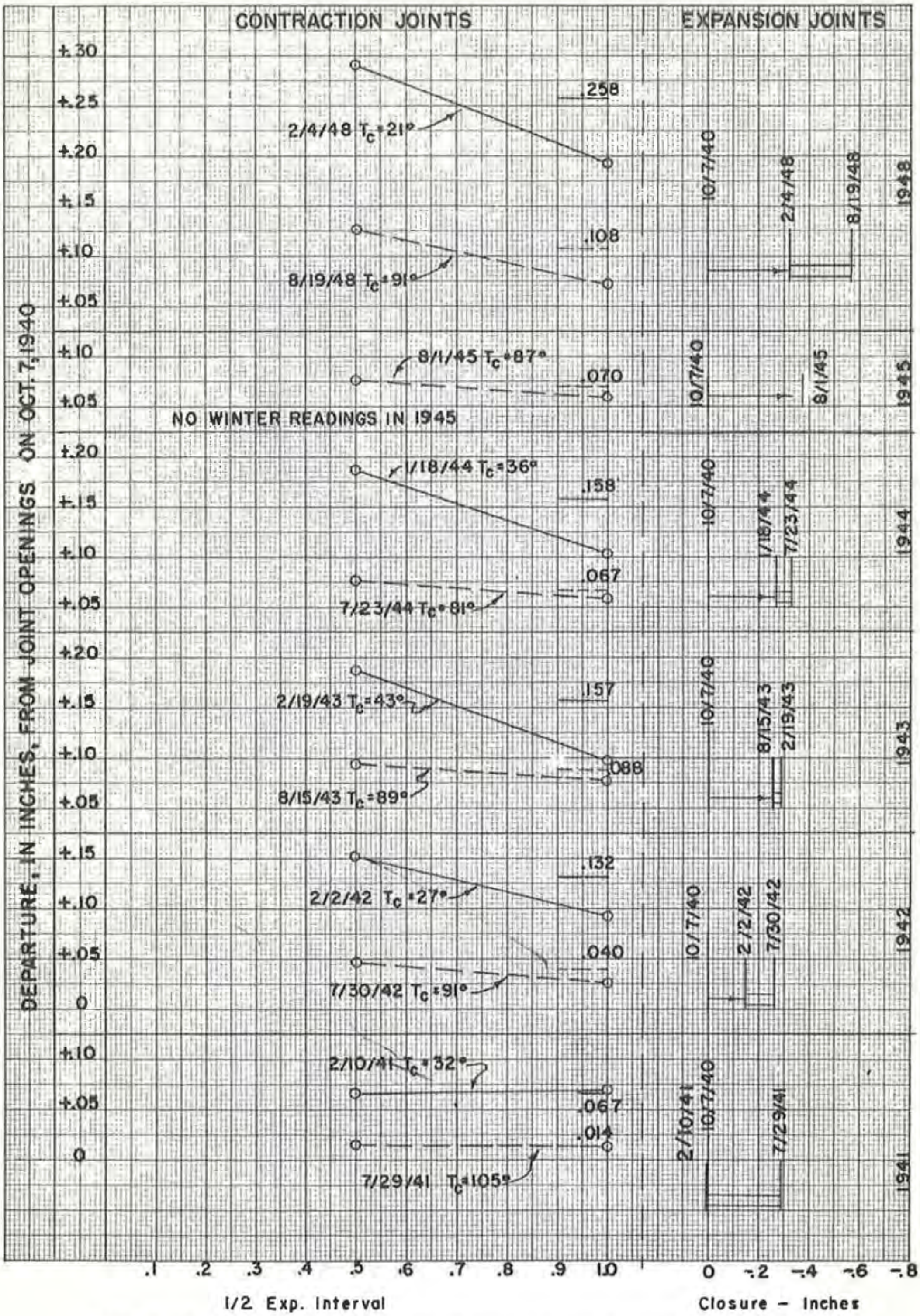


Figure 15. Annual and permanent changes in joint openings, expansion joint interval=120 feet, contraction joint interval=30 feet.

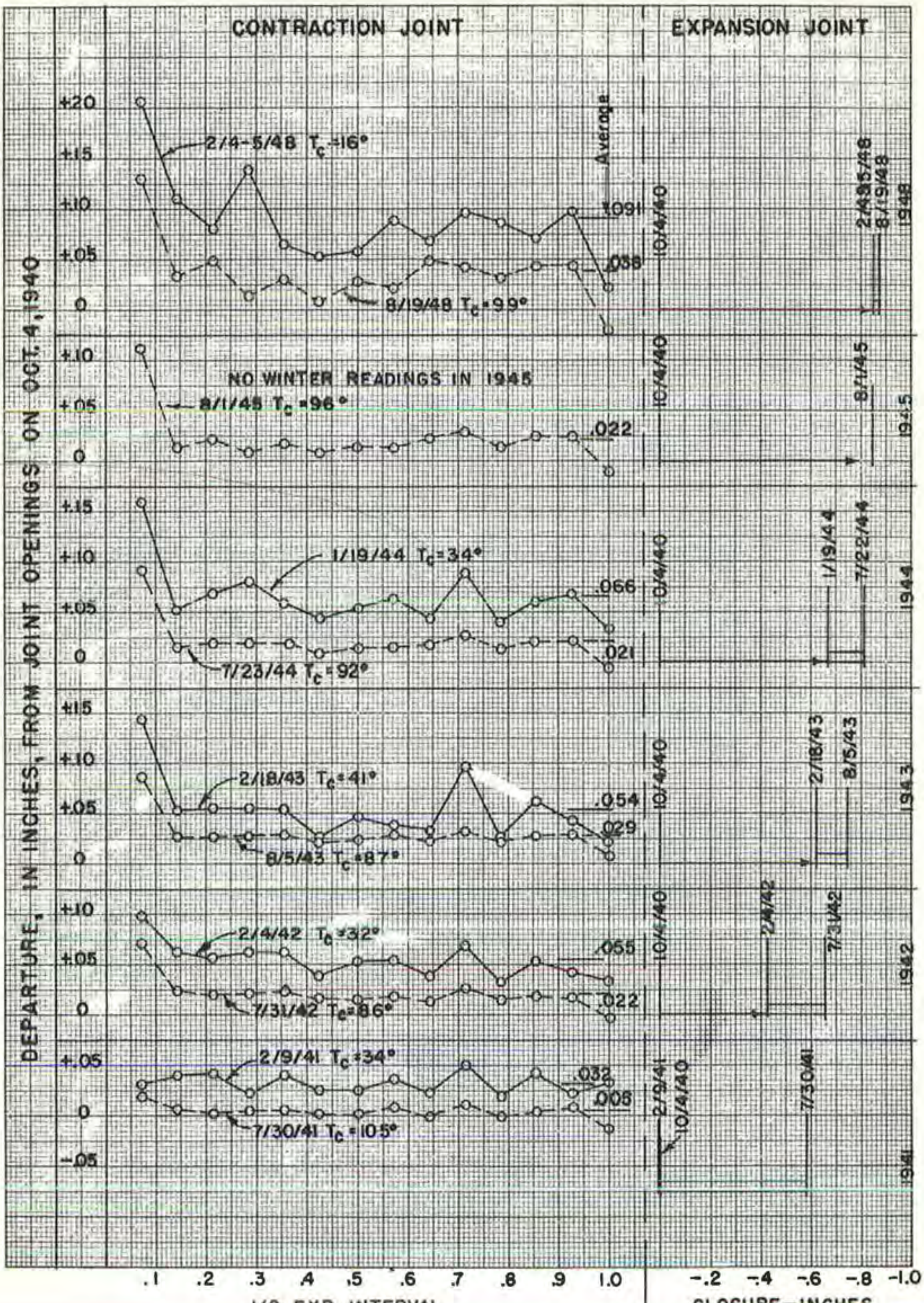


Figure 16. Annual and permanent changes in joint openings, expansion joint interval=420 feet, contraction joint interval=15 feet.

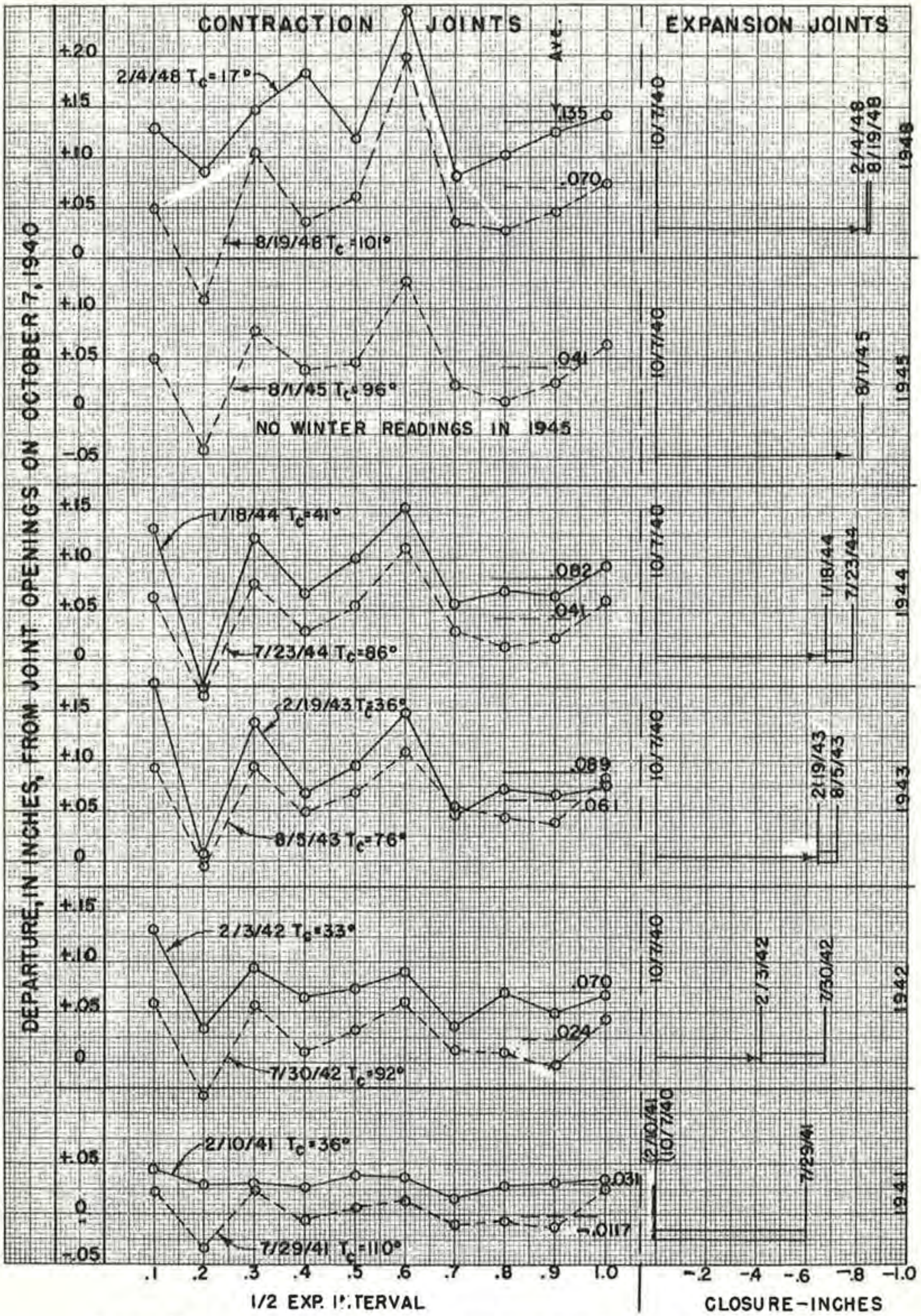


Figure 17. Annual and permanent changes in joint openings, expansion joint interval=400 feet, contraction joint interval=20 feet.

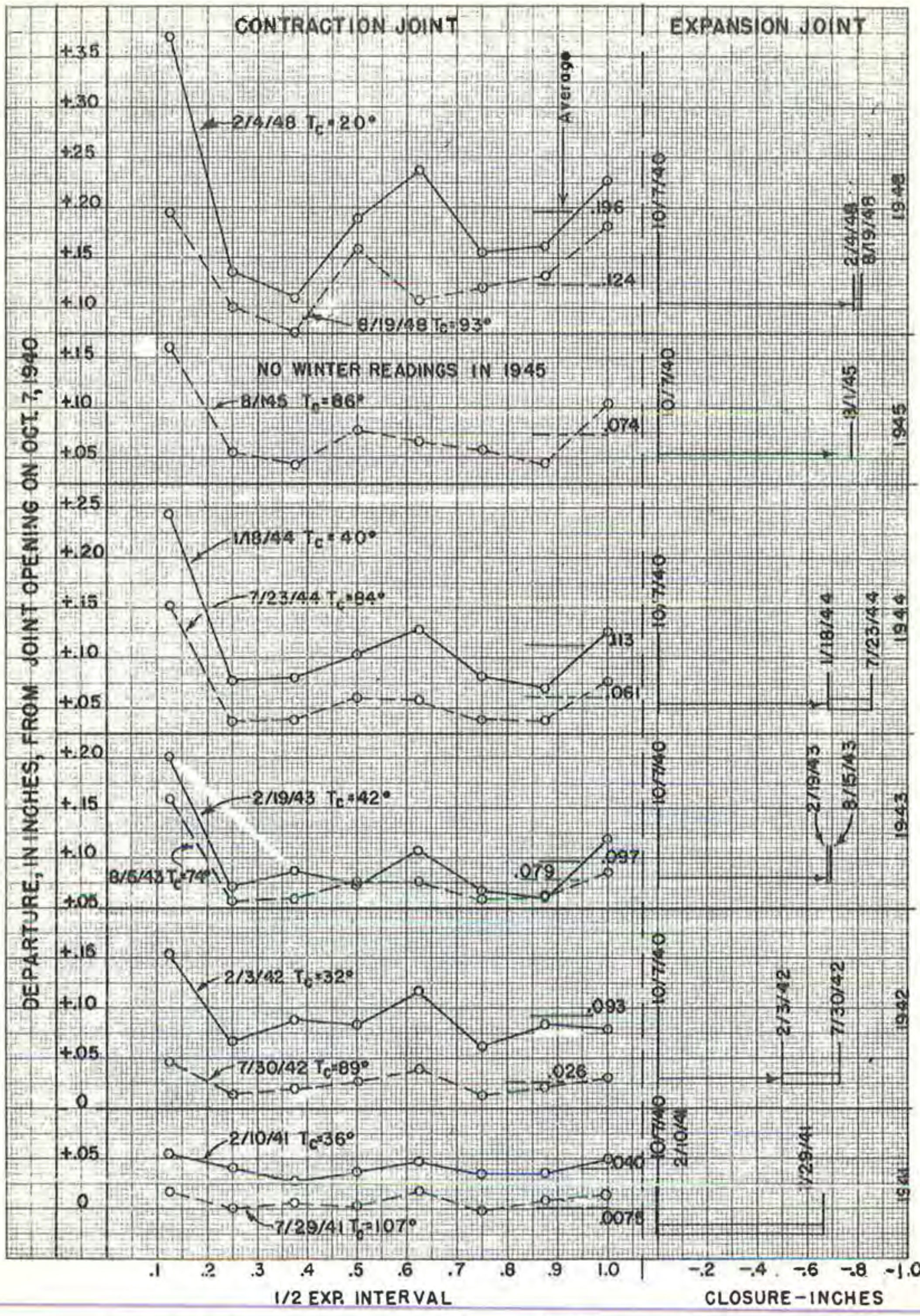


Figure 18. Annual and permanent changes in joint openings, expansion joint interval=400 feet, contraction joint interval=25 feet.

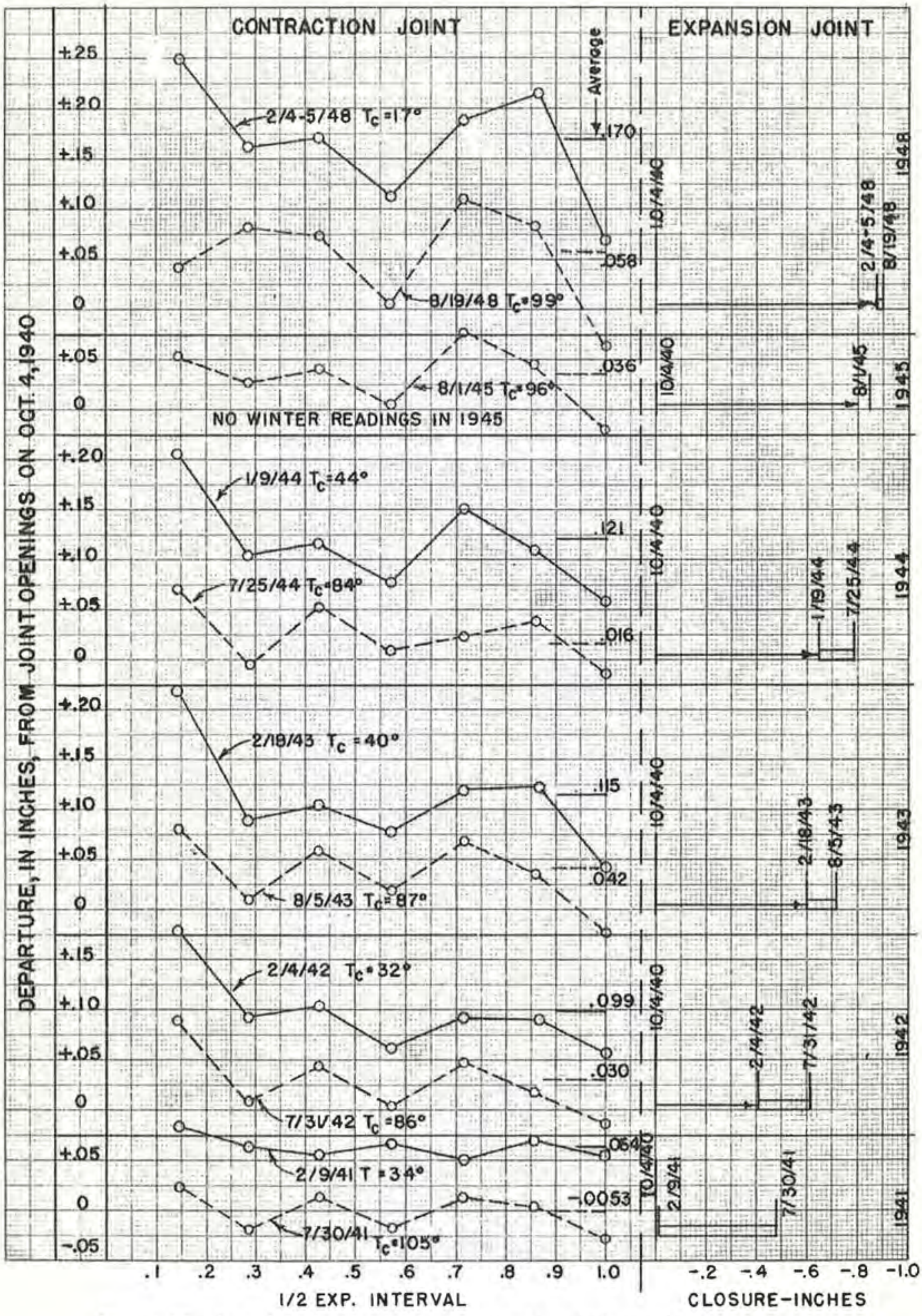


Figure 19. Annual and permanent changes in joint openings, expansion joint interval=420 feet, contraction joint interval=30 feet.

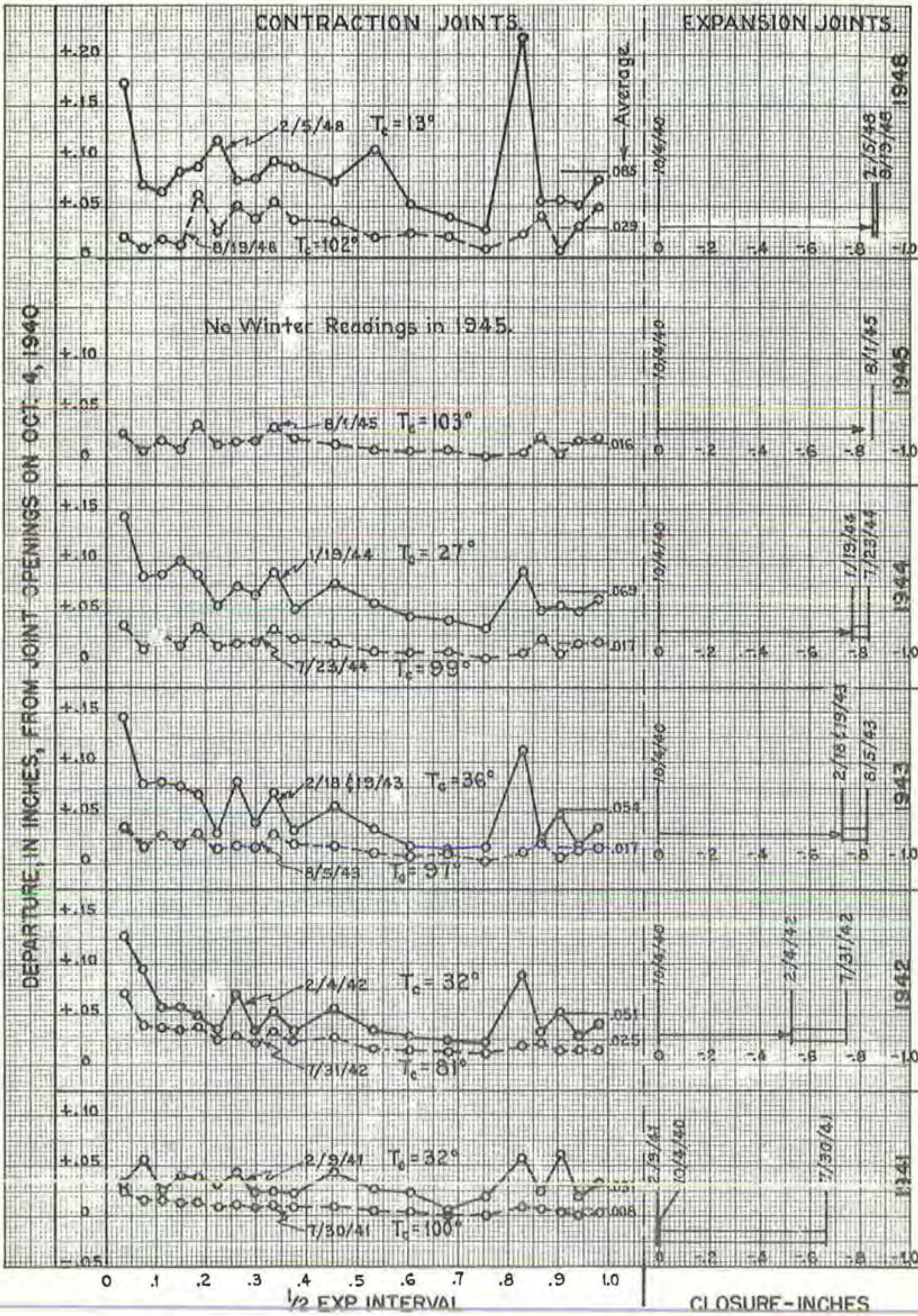


Figure 20. Annual and permanent changes in joint openings, expansion joint interval=795 feet, contraction joint interval=15 feet.

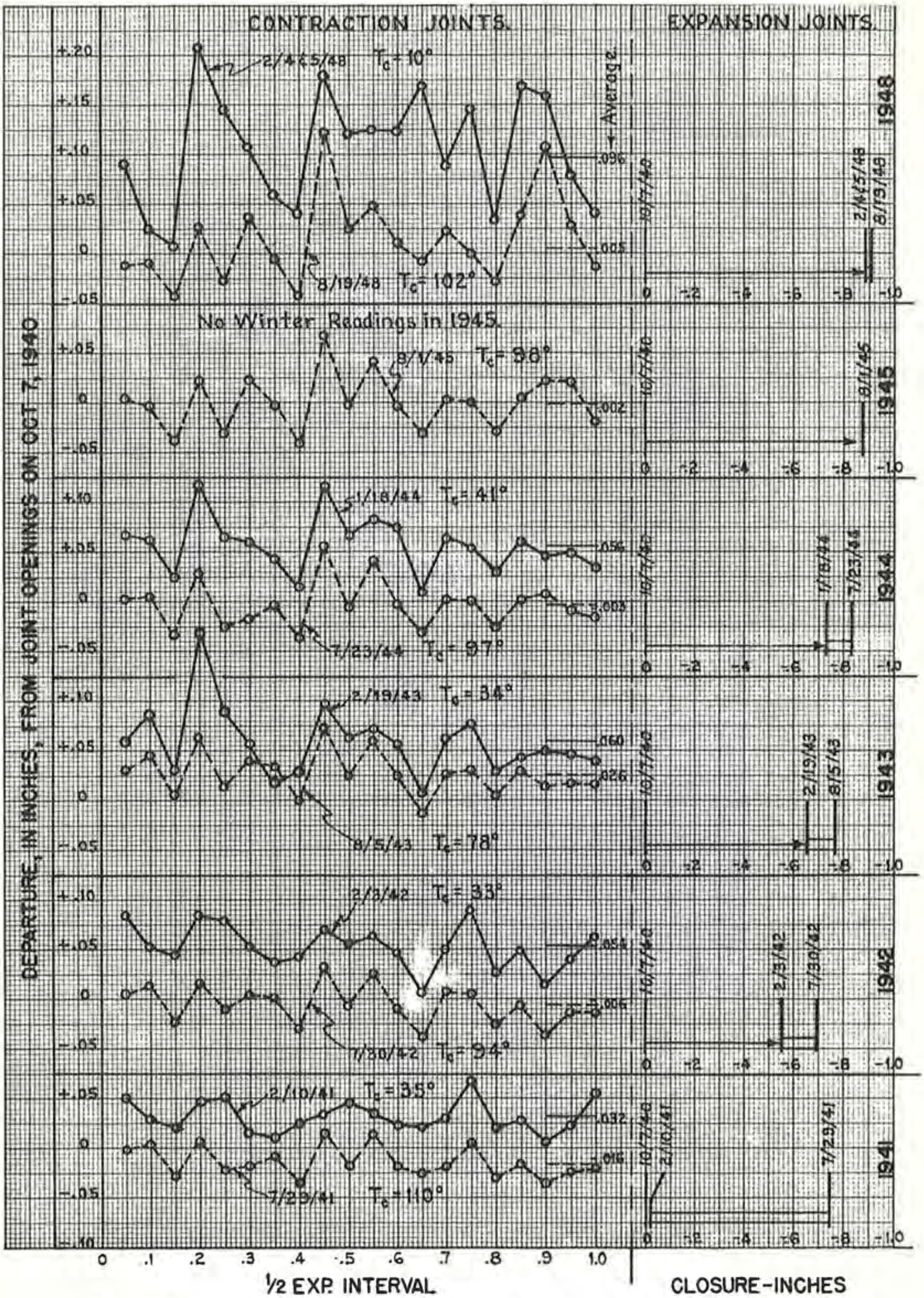


Figure 21. Annual and permanent changes in joint openings, expansion joint interval=800 feet, contraction joint interval=20 feet.

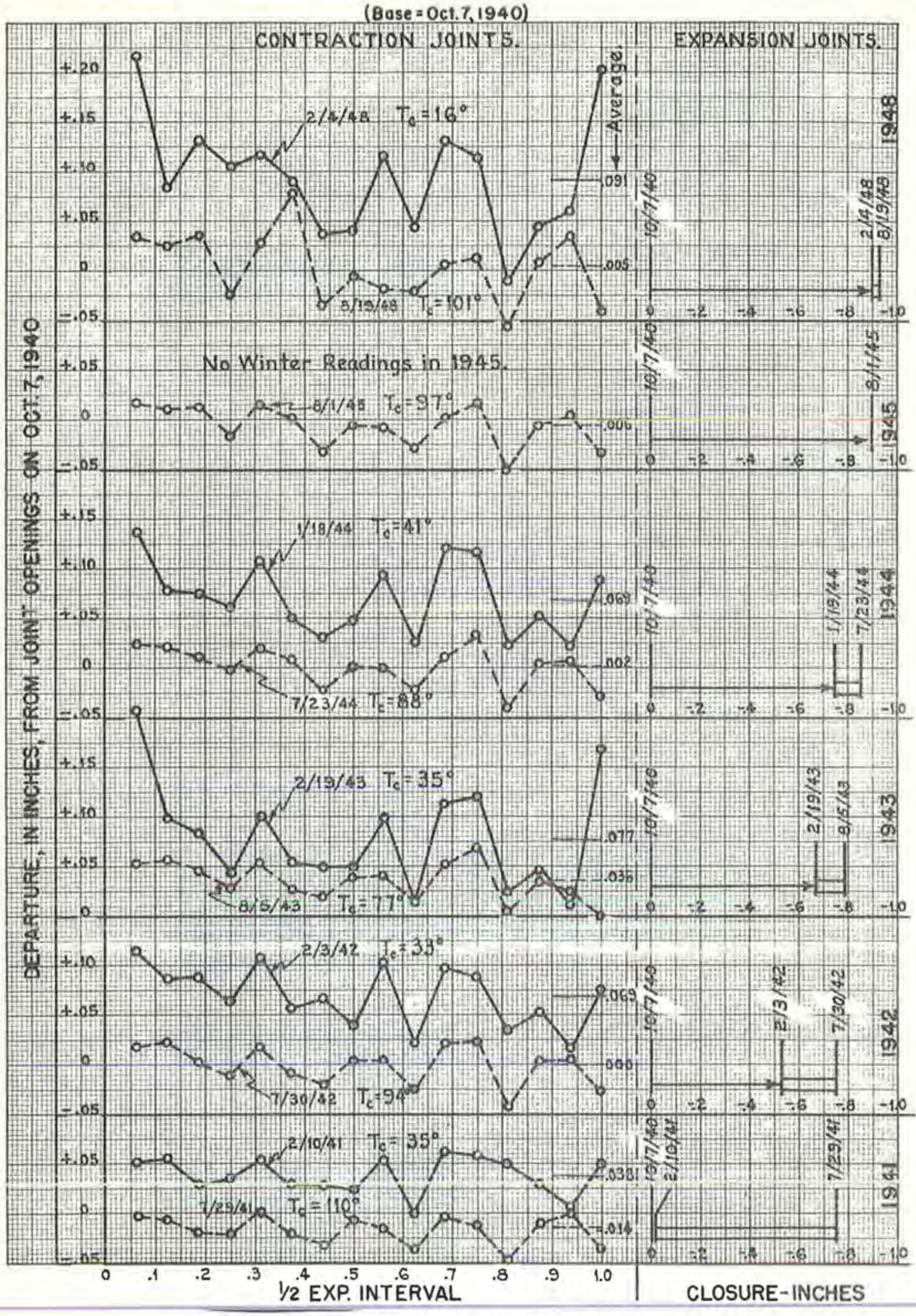


Figure 22. Annual and permanent changes in joint openings, expansion joint interval=800 feet, contraction joint interval=25 feet.

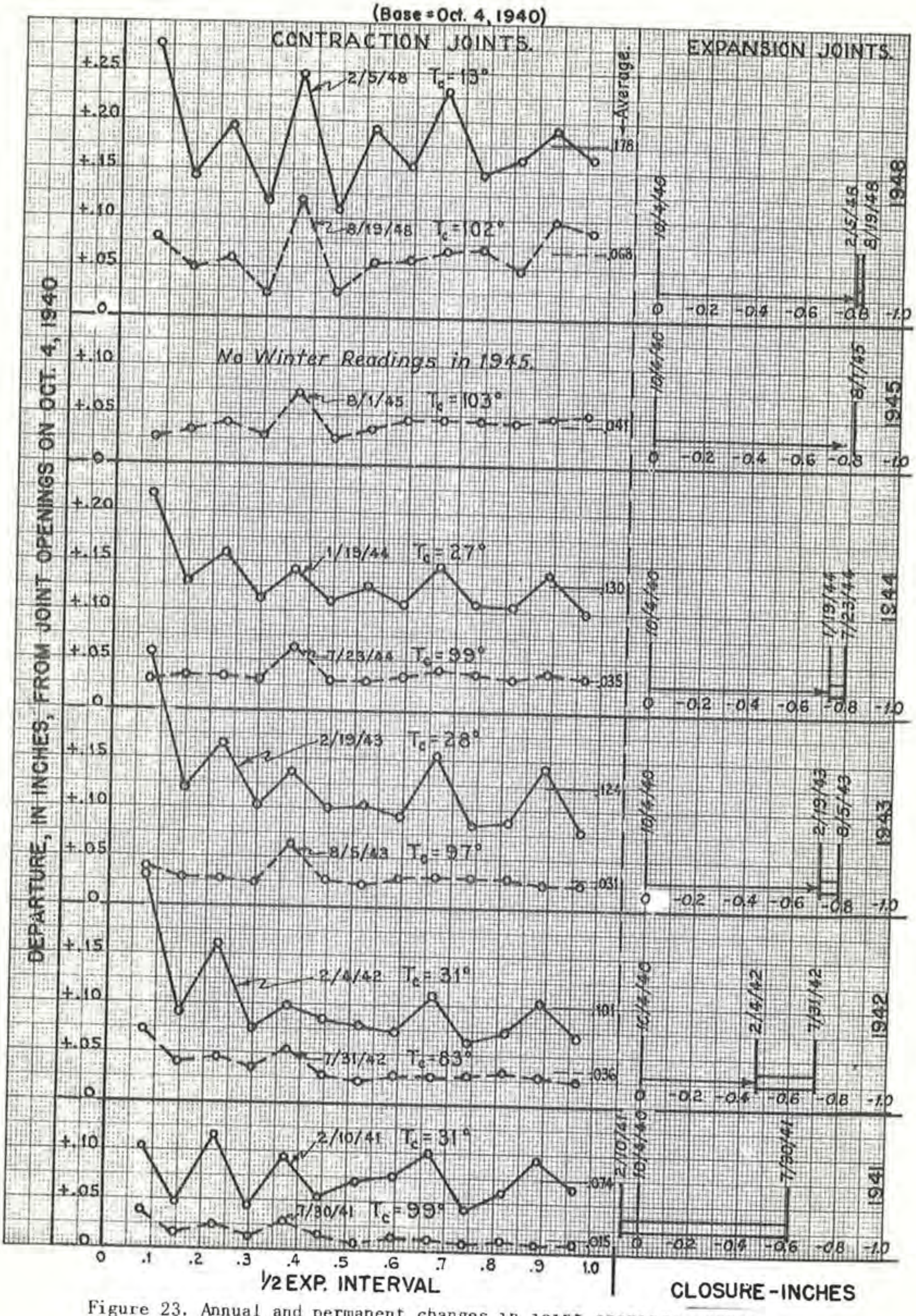


Figure 23. Annual and permanent changes in joint openings, expansion joint interval=810 feet, contraction joint interval=30 feet.

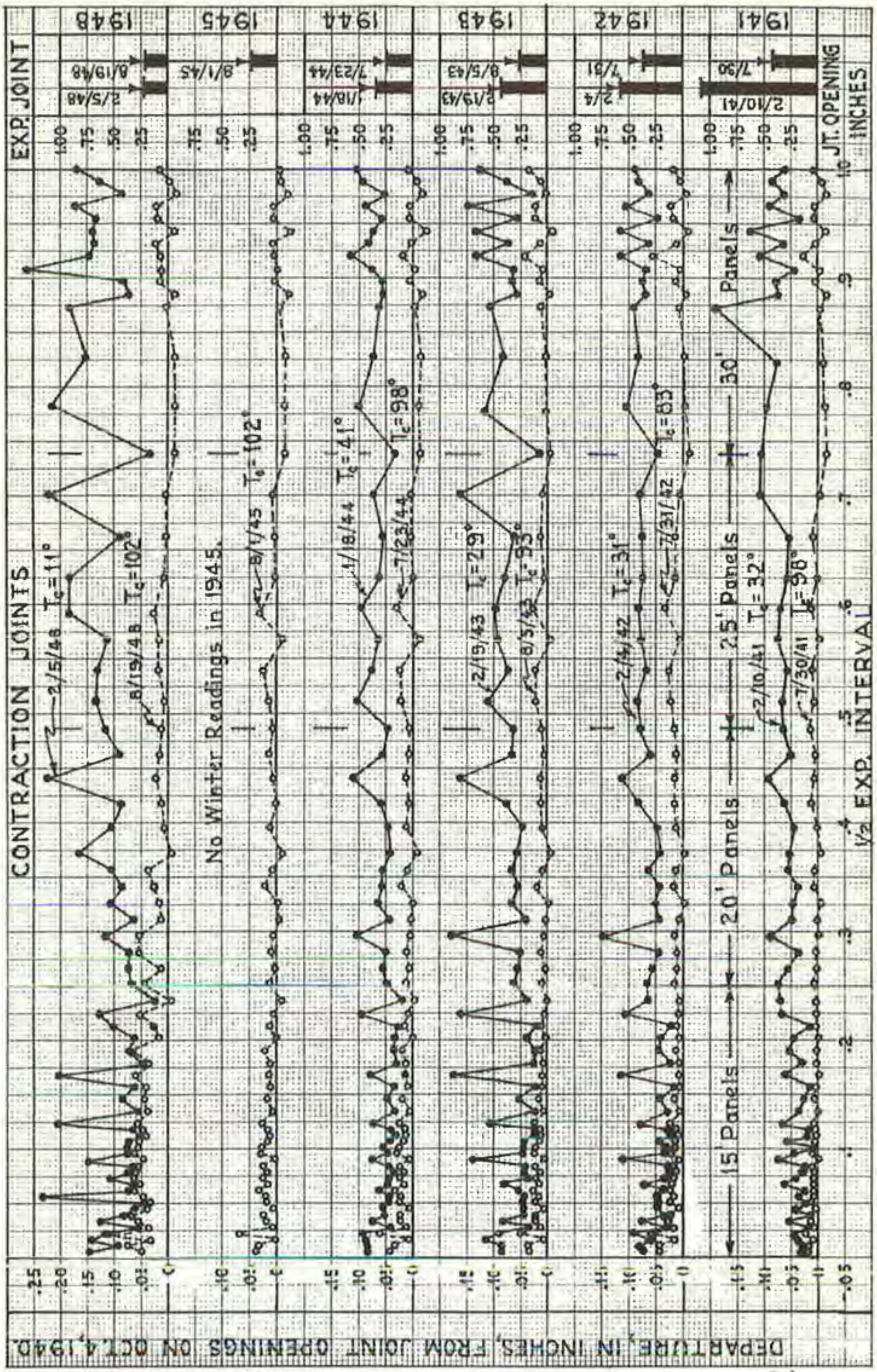


Figure 24. Annual and permanent changes in joint openings. West ½ of div. 9 - Sta. 400 + 51.6 to Sta. 426 + 82.1.

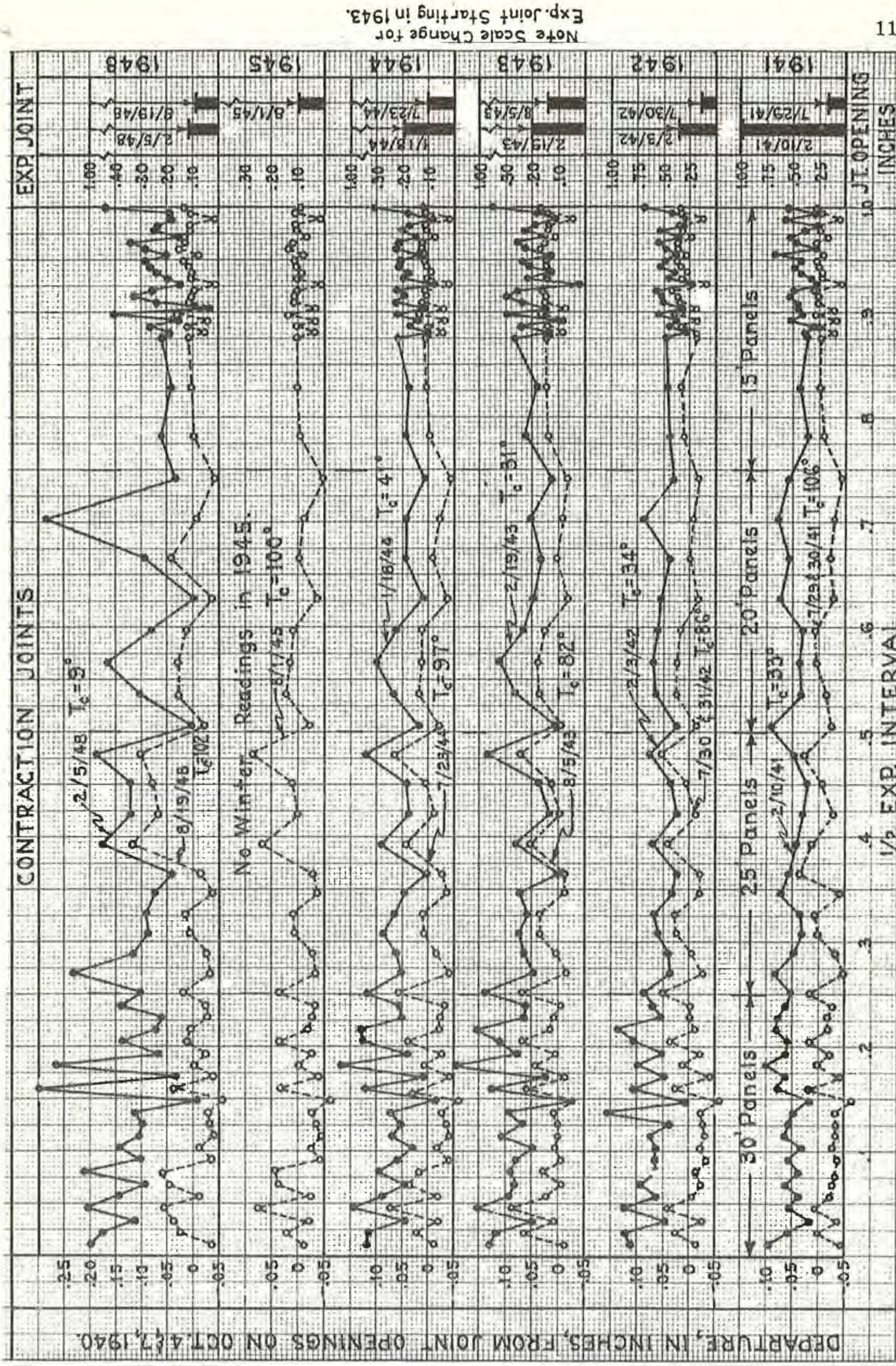


Figure 25. Annual and permanent changes in joint openings. East ½ of div. 9 - Sta. 426 + 82.1 to Sta. 453 + 11.7.

TABLE 5

24-HOUR AVERAGE ANNUAL DAILY TRAFFIC

Vehicle Type	1936-1937	1941	1944	1946	1948	1950	1952	[a]Av. Max. Axle Load
Passenger Cars	280	484	429	515	1081	1215	1623	
Single Unit Trucks								
2-axled	39	73	123	124	142	246	214	7626
3-axled	3	11	37	2	2	5	10	10702
Tractor-semitrailers								15819
3-axled	-	2	10	29	25	55	41	
4-axled	-	-	1	8	18	45	55	
5-axled	-	-	-	1	2	7	4	
Trucks with Trailers	-	-	-	-	-	2	4	8828
Busses	2	2	2	3	4	5	7	
Total Vehicles	324	572	602	682	1274	1580	1958	

[a] Average maximum axle load of loaded vehicles by type based on loadometer data collected at 14 rural trunk highway locations. This data approximates that which existed on the experimental project.

Figures 31 and 32 show the average permanent change in contraction joint openings for 15-foot and 30-foot panels located near the center of Division 9 (1 mile without expansion joints). By 1948 the permanent opening of the contraction joints was less than 0.02 inch for the 15-foot panels and about 0.05 inch for the 30-foot panels.

Figures 33, 34 and 35 show the average permanent change in expansion joint openings for intervals of 420 feet, 800 feet and one mile. In 1948 the closure was nearly equal for all sections, being 0.87 inch, 0.89 inch, and 0.85 inch respectively. Since the time interval between concrete placement and the initial readings varied from two to eight weeks, there could be considerable closure of the expansion joints in the long expansion joint intervals before the first readings were taken. Since there was very little difference between winter and summer readings as taken in 1948, we can consider the expansion joints at the ends of Division 9 as completely closed. In comparison, the average closure of expansion joints at 120-foot intervals (not including the section of 60-foot reinforced panels) was 0.57 inch. Thus, after eight years, the expansion joints spaced 400 feet or more apart have permanently closed 85 to 90 percent of their original one inch width.

Figure 36 shows the rate of this permanent closure since construction, in relation to the 1948 summer closure, for the various expansion intervals. The rates of closure for the sections having one mile and 800-foot spacing of expansion joints were very nearly the same, being very rapid in the first two years after construction. The expansion joints on the 420 foot sections closed almost as fast as those on the longer sections and, after eight years, had closed an equal amount. The expansion joints at 120-foot intervals closed at a much slower rate and can be expected to continue closing as dirt infiltrates the contraction joints.

Figure 37 shows the progressive closure of expansion joints and cumulative opening of contraction joints for typical 400 foot expansion intervals and various panel lengths.

A summary of average contraction joint openings in the winter and summer of 1948 is shown in Figure 38 for all combinations of expansion and contraction joint spacings. This shows the advantage of both short panels and wide spacing or omission of expansion joints. The smaller summer openings of contraction joints are associated with the longer expansion intervals and the smaller winter openings are associated with the shorter panel lengths. The expansion intervals of 420 feet, 795 feet and one mile with 15-foot panels show the least winter opening and minor summer opening of the contraction joints. This indicates that the rate of infiltration of foreign material into these

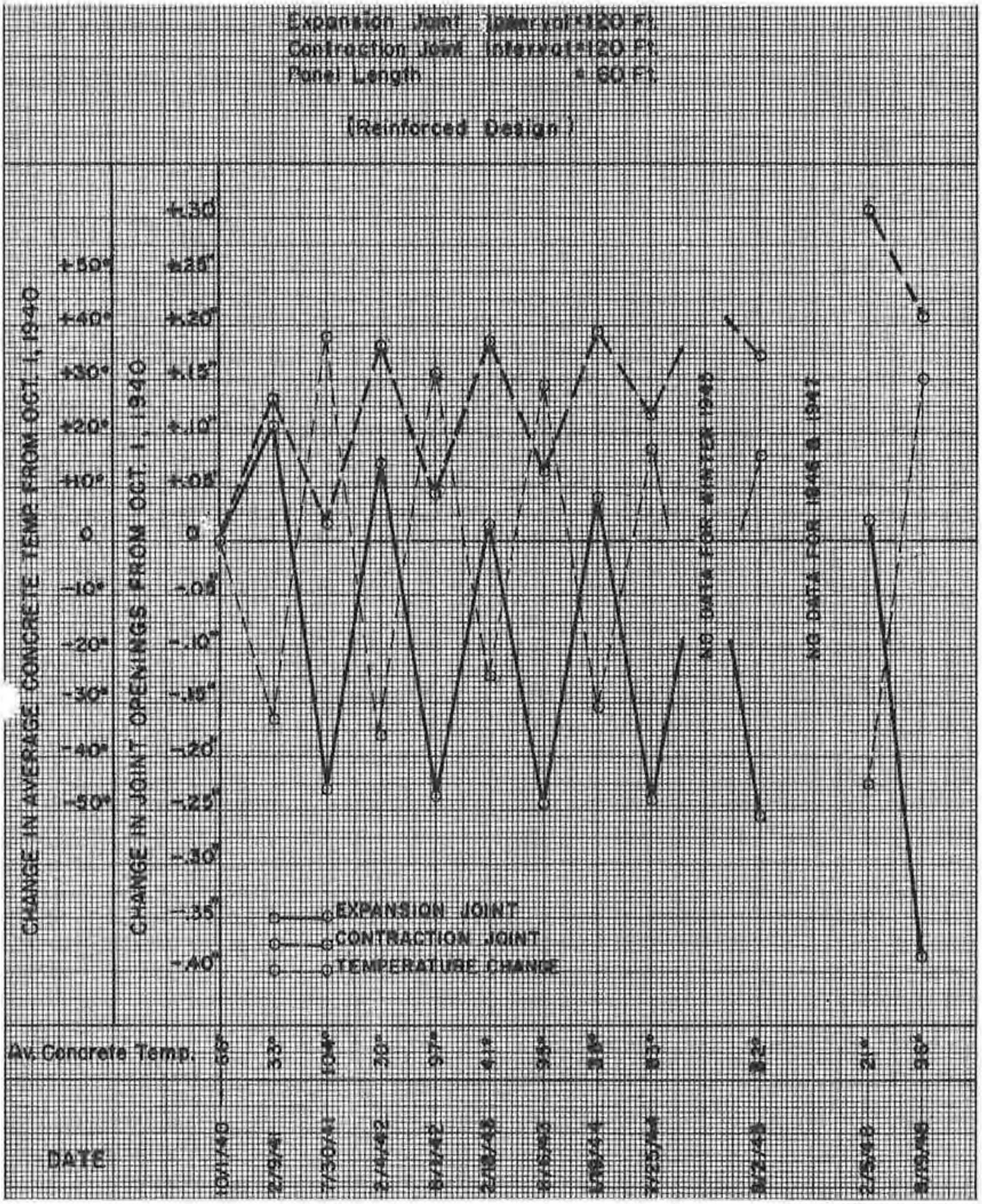


Figure 26. Annual and permanent changes in joint openings.

joints has been negligible and that possible future infiltration is less likely than on other sections.

SEASONAL MOISTURE CHANGE AND SHRINKAGE

A considerable number of gypsum blocks were installed during construction for use in determining changes in the moisture content of the concrete and subgrade soil. These blocks and the method of moisture determination in which they are used were developed

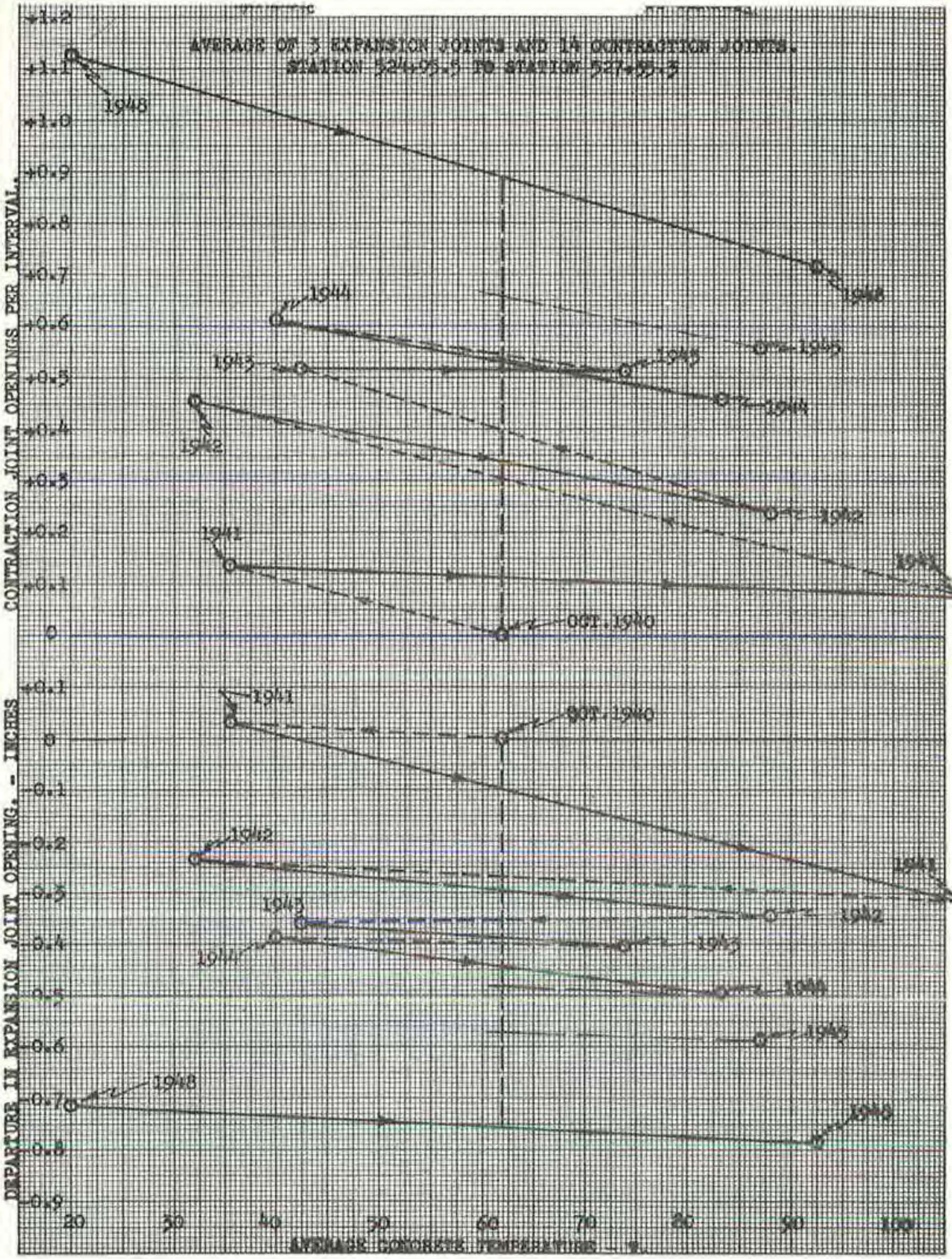


Figure 27. Annual and permanent changes in joint openings. Expansion interval of 120 feet - 15 foot panels.

by Dr. G. J. Bouyoucos and are described in Technical Bulletin No. 172 of the Michigan State College, April, 1939.

The data obtained by the moisture blocks was not entirely satisfactory. A general but slow increase in moisture was indicated in the subgrade soils, although a detailed analysis of the data was not made. The blocks installed in the concrete were unsatisfactory, because they did not give consistent or well defined readings on the wheatstone bridge.

Although it was not possible to determine seasonal moisture changes by the Bouyoucos method, the effect of these changes were secured from a series of measured changes in length of a section of pavement on which measurements were taken at the time of initial hardening and at various times thereafter. The measured section was located between expansion joints at Sta. 524+95.5 and Sta. 526+15.4 and consisted of a series of 15-foot

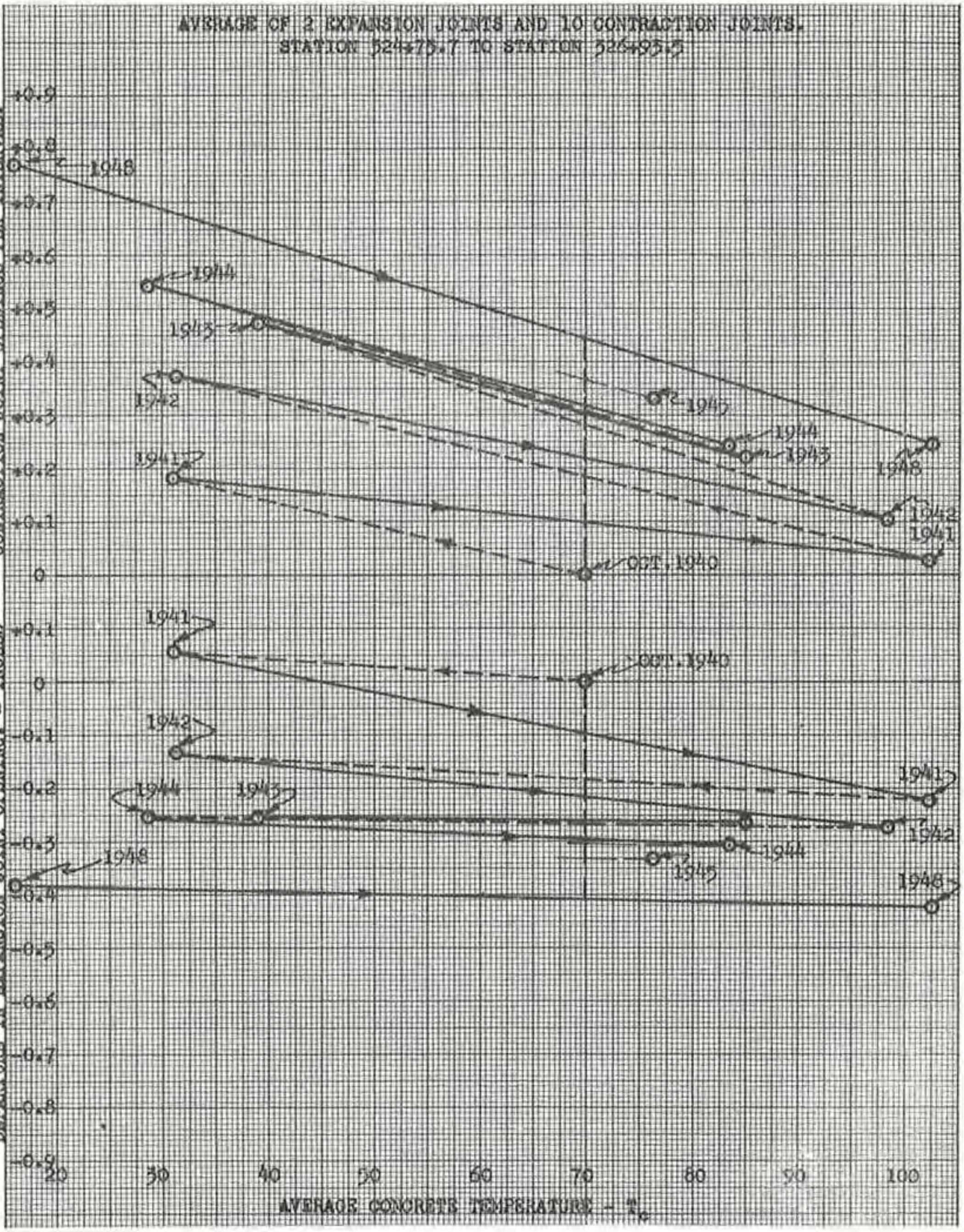


Figure 28. Annual and permanent changes in joint openings. Expansion interval of 120 feet - 20 foot panels.

panels. The total length, center to center of expansion joints, as built was 119. 9 feet. Extensometer points were set across every joint in this section, including the expansion joints at both ends. This section was one of a series of sections of approximately the same length; i. e., 120 feet, so that it has not been affected by unbalanced forces from adjacent sections of dissimilar length.

Table 6 shows the detailed data relative to these measurements and the computations of the values in each column are explained by footnotes. The initial readings, on September 11, 1940, were taken as soon after the placement of the concrete as hardening would permit. This was after the brooming of the surface and before the curing

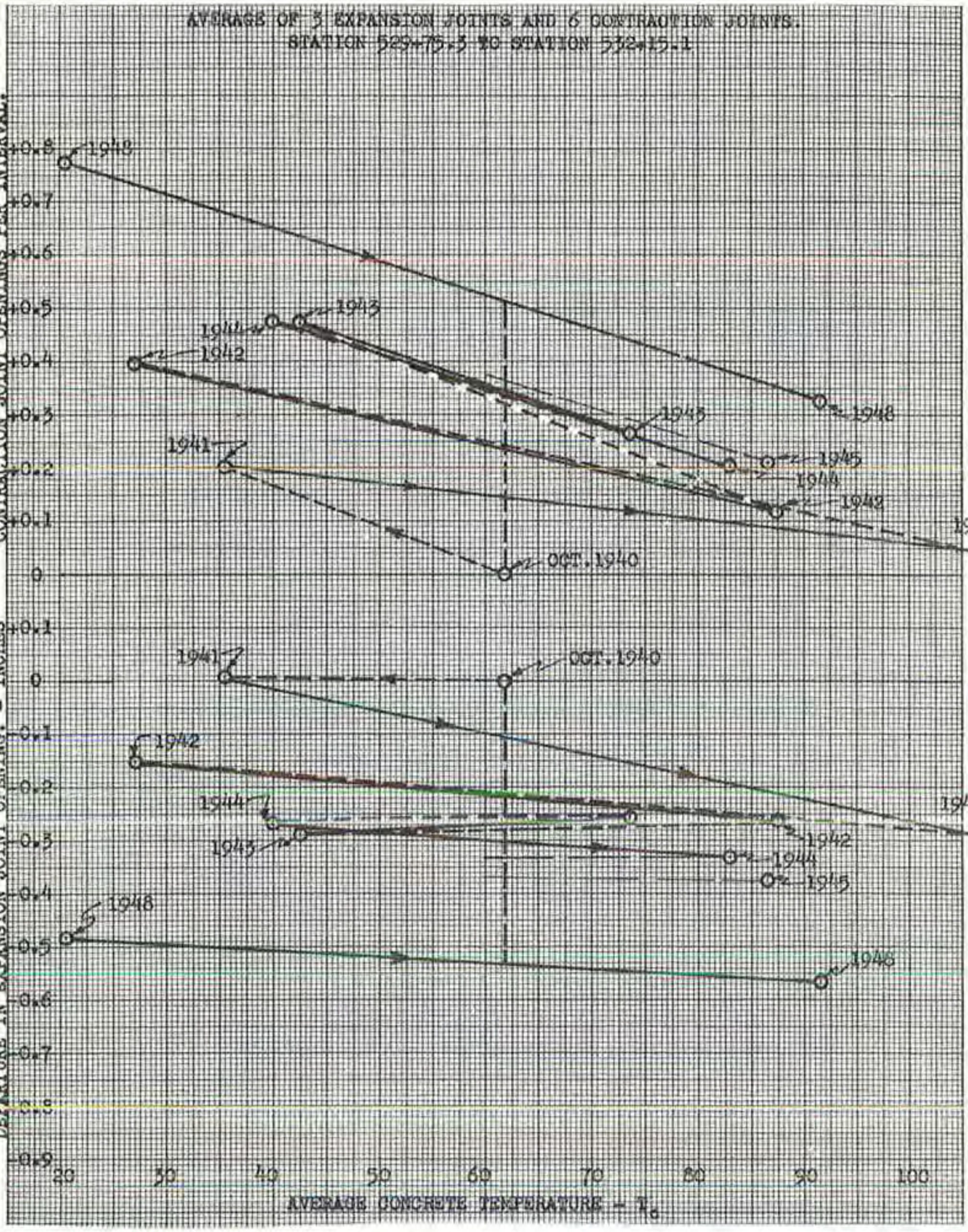


Figure 29. Annual and permanent changes in joint openings. Expansion intervals of 120 feet - 30 foot panels.

TABLE 6

DATA FOR COMPUTING EFFECT OF SHRINKAGE & SEASONAL MOISTURE VARIATION

SECTION: STA. 524+99.5 to STA. 526+15.4
Exp. Jt. Interval = 119.9' = 1438.8"
Cont. Jt. interval = 15'

DATE	GAGE LENGTHS - INCHES	LENGTH OF MEASURED CONCRETE - INCHES (3)	OVERALL CORRECTED LENGTH OF SECTION - INCHES (4)	CUMULATIVE CHANGE IN SECTION LENGTH from 9/11/40 - INCHES (5)	SEASONAL CHANGE IN LENGTH - INCHES (6)	CONT. JT. OPENINGS: CUMULATIVE CHANGE from 9/11/40 (7)	CONT. JT. OPENINGS: SEASONAL CHANGE - INCHES (8)	EXP. JT. OPENINGS: CUMULATIVE EXP. SPACE from 9/11/40 (9)	EXP. JT. OPENINGS: SEASONAL CHANGE (10)	TEMP. CONC. (11)	CHANGE IN TEMP. (12)
(1)	A	B	C	D	E	F	G	H	I	J	K
9/11/40	80.0110	1358.7890	1437.8000	.0000	.0000	.0000		1.0000		54.0(2)	
10/ 7/40	80.0810	1358.7190	1437.7259	-.0741	-.0741	+.0586	+.0586	1.0155	+.0155	61.5	+ 7.5
2/10/41	80.2425	1358.5575	1437.5550	-.2450	-.1709	+.2020	+.1434	1.0430	+.0275	36.2	-25.3
7/29/41	79.8340	1358.9660	1437.9873	+.1873	+.4323	+.1269	-.0751	.6858	-.3572	106.8	+70.6
2/ 3/42	80.3135	1358.4865	1437.4799	-.3201	-.5074	+.5666	+.4397	.7535	+.0677	32.0	-74.8
7/30/42	79.9845	1358.8155	1437.8280	+.0280	+.3481	+.3177	-.2489	.6543	-.0992	88.0	+56.0
2/19/43	80.2615	1358.5385	1437.5349	-.2651	-.2931	+.6159	+.2982	.6492	-.0051	42.2	-45.8
8/ 5/43	80.2300	1358.5700	1437.5683	-.2317	+.0334	+.6273	+.0114	.6044	-.0448	73.8	+31.6
1/18/44	80.3535	1358.4465	1437.4376	-.3624	-.1307	+.7326	+.1053	.6298	+.0254	40.1	-33.7
7/23/44	80.1150	1358.6850	1437.6900	-.1100	+.2524	+.5783	-.1543	.5317	-.0981	83.2	+43.1
8/ 1/45	80.1575	1358.6425	1437.6450	-.1550	-.0450	+.7086	+.1303	.4464	-.0853	87.0	+ 3.8
2/ 4/48	80.6480	1358.1520	1437.1260	-.6740	-.5190	+1.3549	+.6463	.3191	-.1273	20.0	-67.0
8/19/48	80.1490	1358.6510	1437.6540	-.1460	+.5280	+.8961	-.4588	.2499	-.0692	92.5	+72.5

(1) Placement Date. (2) Hardening Temp. of Concrete. (3) B=1438.8-A. (4) $C=(1438.8-1.0) \times \frac{B}{1358.789} = \frac{1437.8}{1358.789} \times B = 1.0581481B$ (5) D. = differences between values in Column C and initial length. (6) E. = Algebraic differences between successive values in Column D. (7) Summation of all contraction Joint Openings, as measured and corrected for panel length: gage length ratio. (8) Differences between successive values in Column F. (9) Same as note (7). (10) Same as note (8). (11) Measured average temperature of pavement slab. (12) Differences between successive values in Column J.

blankets were applied, which was about two hours and 30 minutes after the concrete was placed on the grade. The next measurement was made about one month later and then measurements were taken in the summer and winter of each year up to and including 1944. Thereafter only three sets of measurements were made, in the summer of 1945 and winter and summer of 1948. Seasonal changes in the length of this section are shown in Column E and the corresponding changes in average concrete temperature are shown in Column K. The data in these two columns are transferred to Table 7 for use in further computations.

The thermal coefficients shown in Table 7, Column E, are corrected values taken from the curve; the corrections being made for the center of each season's range in temperature.

With the initial readings and hardening temperature as a base, the theoretical departures in length for the seasonal changes in temperature are shown in Table 7, column F. Corresponding actual measured changes in length are shown in Column G. The differences between these values, Column H, indicate the seasonal variations due to moisture and shrinkage. These data are shown graphically in Figure 39. The lower graph in this figure shows the departures in concrete temperature from the initial hardening temperature of 54^0 F. at various times when measurements were made up to a total age of 2,899 days, which was August 19, 1948. The upper graph shows, for the same ages, the departures from the initially measured length of both the theoretical thermal change and the actual measured change. It will be observed that the first measurement after placement (age 26 days) shows a decrease in length of -.0741 inch, while the difference in concrete temperature increased 7.5^0F. above the hardening temperature which is equal to a theoretical thermal increase in length of +0.0666 inch. The total of these two departures (0.1407 inch) represents largely the effect of initial or early shrinkage.

Beginning with February 10, 1941 and continuing to July 23, 1944, the differences in length for each period, as shown in Column H, represent principally the effect of seasonal moisture changes on the length of the section. The average of these values is $\pm$ 0.1575 inch which, on the basis of an original length of 1437.8 inches, represents a seasonal moisture coefficient of $\pm$ 0.00011 which operates in opposite direction to seasonal temperature changes. It may be noted that the value in Column H for the period from February to August, 1943 is considerably less than for the other periods. Reference to Figure 2 provides a probable reason for this low value since it shows that the spring of 1943 was unusually dry in comparison with the other years. The data between July, 1944 and August, 1948 were obtained at intermittant periods and do not show true seasonal changes. For this reason, the use of this data is limited in regard to the following calculations.

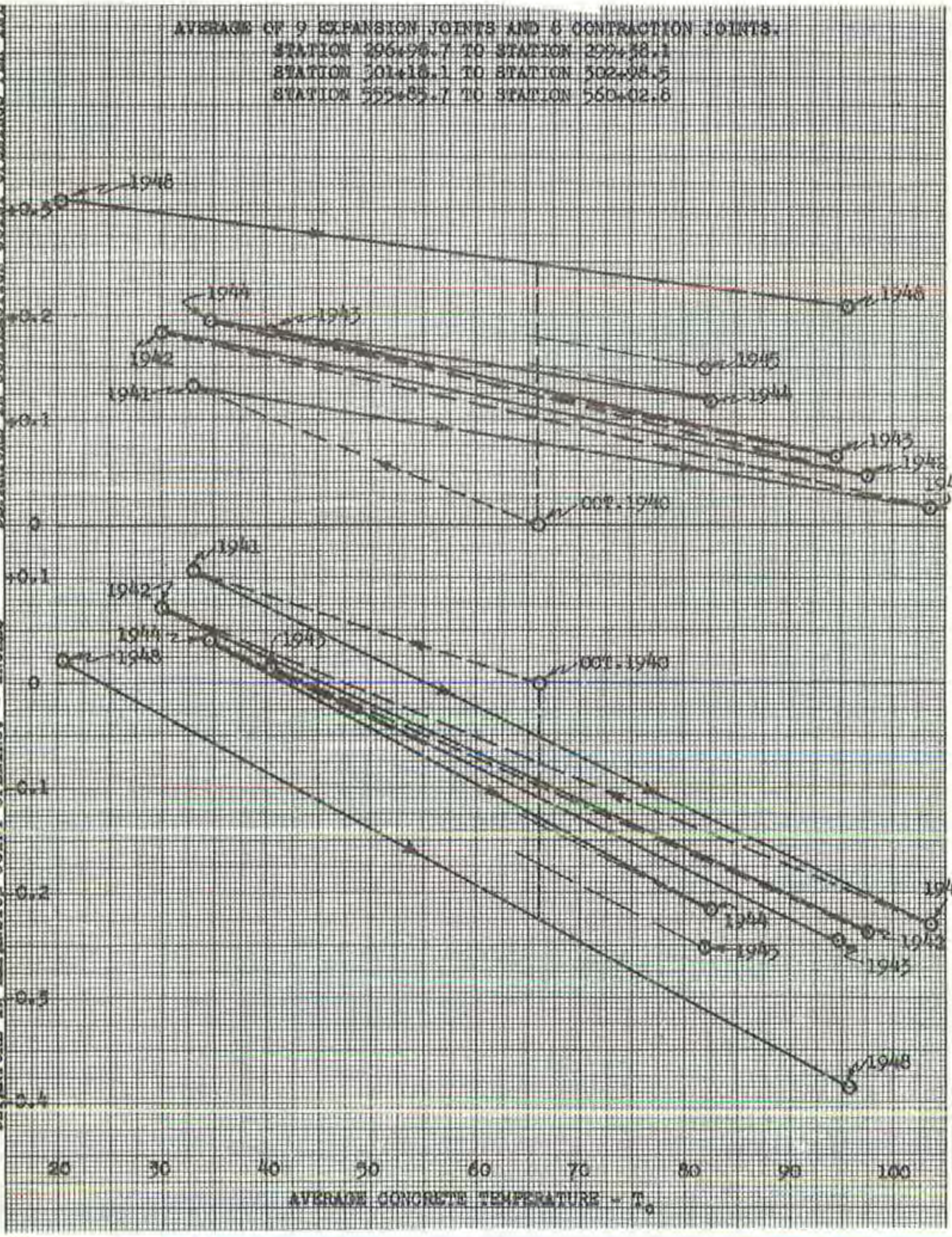


Figure 30. Annual and permanent changes in joint openings. Expansion interval of 120 feet - 60 foot reinforced panels.

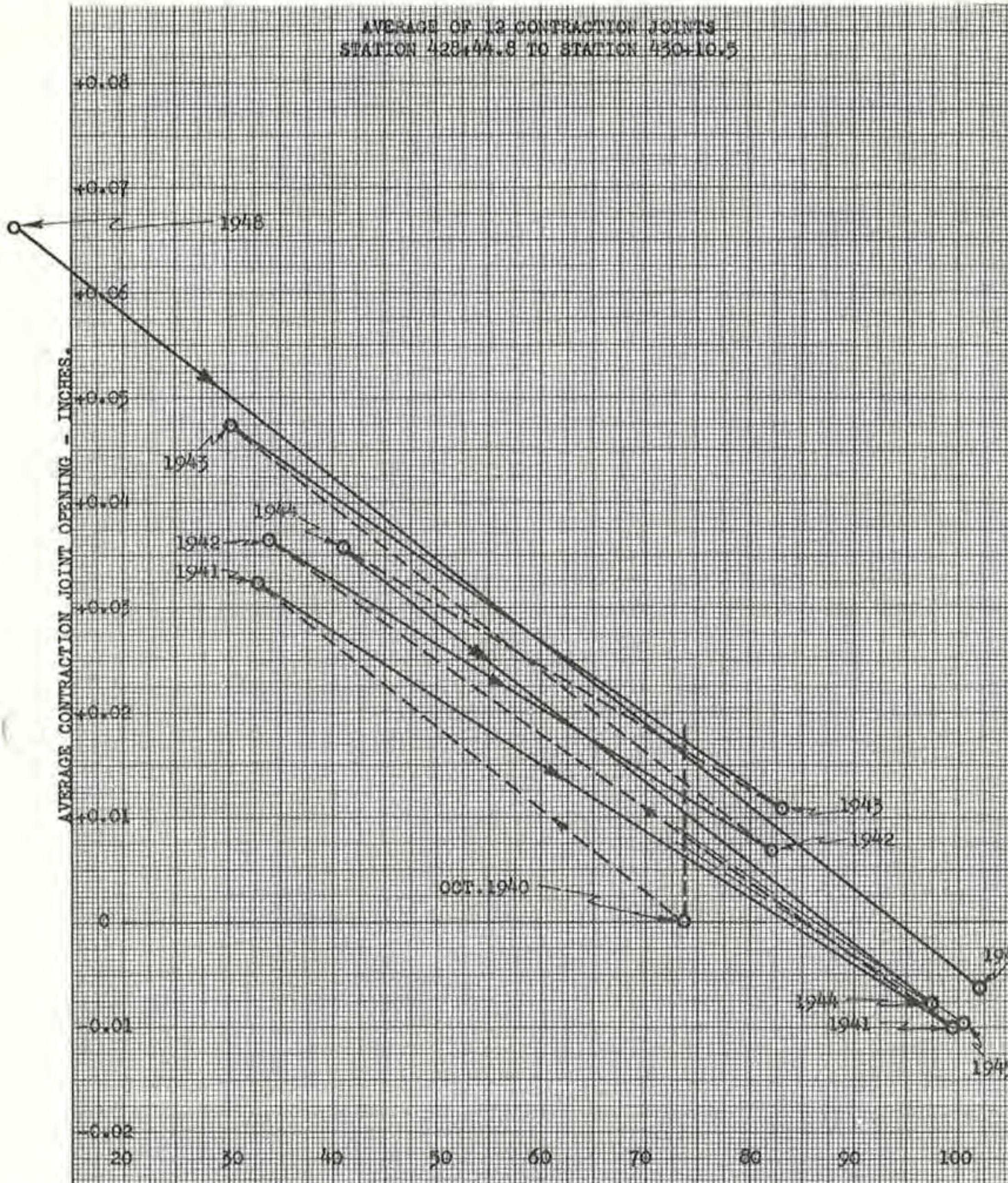


Figure 31. Annual and permanent changes in joint openings. Contraction joint interval of 15 feet near center of division 9.

The sum of the algebraic totals of Columns F and G represent the total difference between the theoretical thermal change and the actual change in length as measured, both accumulated from the initial values. As such, on July 23, 1944 the value of 0.3620 inch represents the total shortening of the section length due to initial and subsequent shrinkage plus the seasonal change in length due to moisture changes. Subtracting the average change in length of 0.1575 inch due to seasonal moisture variations from this total value of 0.3620 inch leaves 0.2045 inch which represents the accumulated shrinkage to July 23, 1944.

These factors, representing changes in length due to moisture variation and shrinkage, may be expressed in terms of equivalent thermal changes as follows:

For Seasonal Moisture Variation:

$$\text{Equivalent thermal change} = \frac{0.1575}{1437.8 \times .00000612} = 17.9^{\circ}\text{F.}$$

For Accumulated Shrinkage:

$$\text{Equivalent thermal change} = \frac{0.2045}{1437.8 \times .00000612} = \underline{23.2^{\circ}\text{F.}}$$

$$\text{Total} = 41.1^{\circ}\text{F.}$$

Thus, on July 23, 1944, approximately four years after construction, there existed on this section a permanent and seasonal compensatory change in length equivalent to that which would be required for a temperature rise of 41.1°F. above hardening tem-

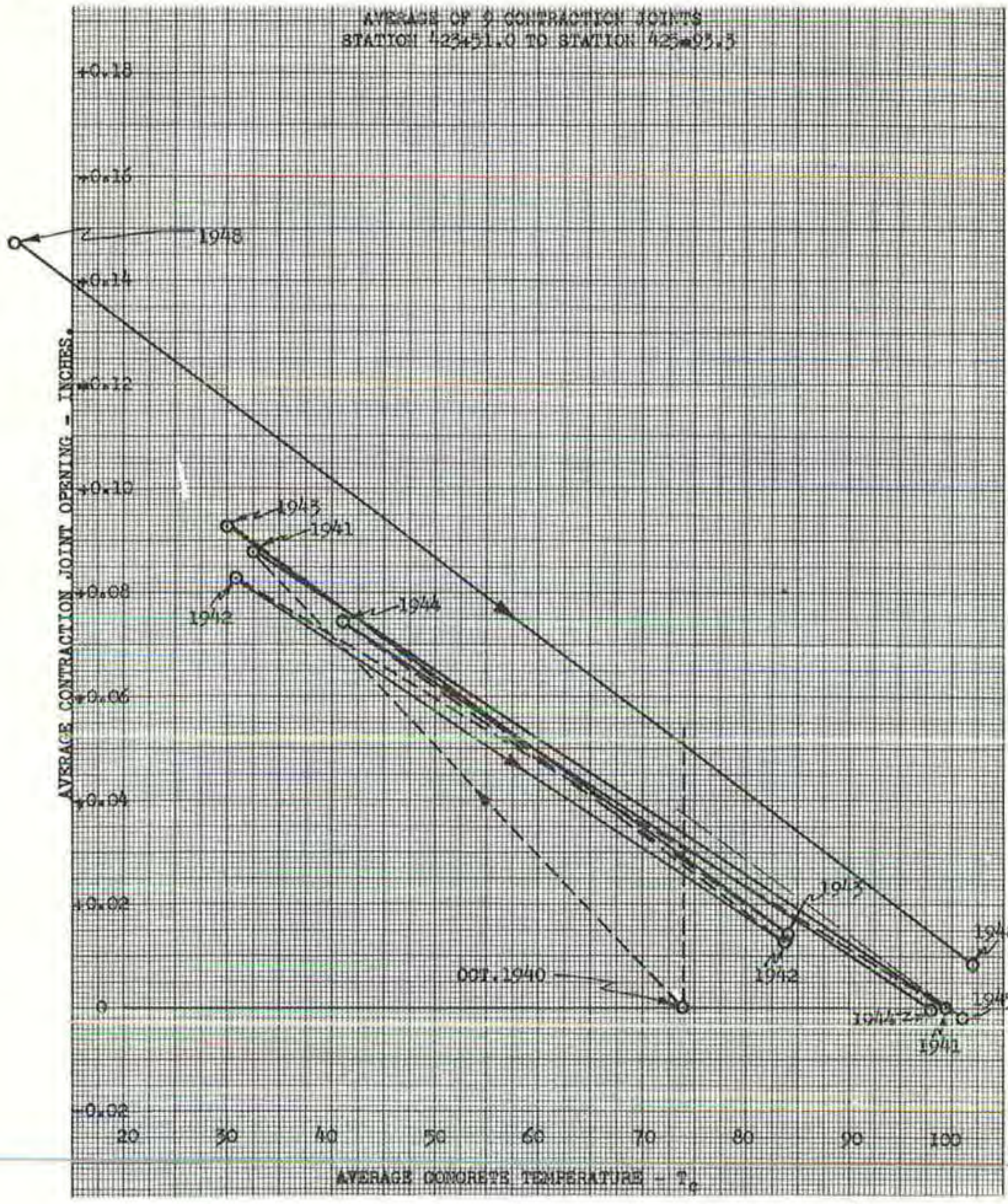
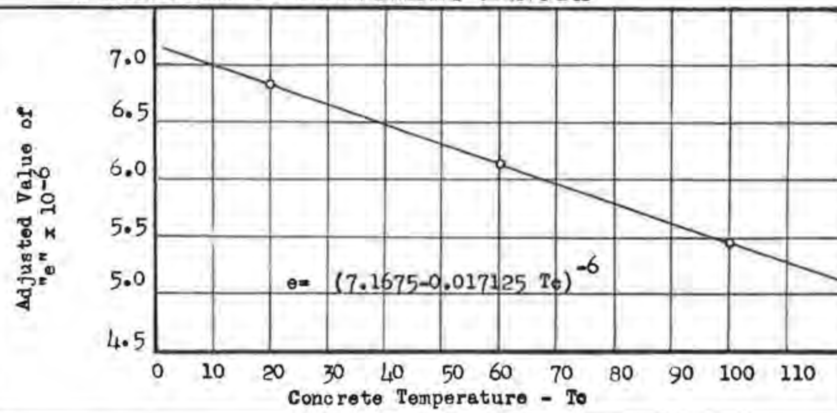


Figure 32. Annual and permanent changes in joint openings. Contraction joint interval of 30 feet near center of division 9.

TABLE 7

EFFECT OF SEASONAL CONCRETE MOISTURE CHANGES & SHRINKAGE ON PAVEMENT LENGTH

TEMPERATURE CORRECTION FOR THERMAL COEFFICIENT "e"



DATA USED IN COMPUTATIONS

PERIOD		AGE - DAYS	CHANGE IN CONC. TEMP. - °F	THEOR. "e" ADJUSTED FOR TEMP. (1)	LENGTH CHANGE IN 120 FT.		
FROM	TO				THEOR. THERMAL CHANGE	ACTUAL MEASURED CHANGE (2)	DIFF.
(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)
9/11/40	10/7/40	26	+7.5	$+6.18^{-6}$	+.0666	-.0741	-.1407
10/7/40	2/10/41	152	-25.3	-6.33^{-6}	-.2303	-.1709	+.0594
2/10/41	7/29/41	321	+70.6	$+5.95^{-6}$	+.6040	+.4323	-.1717
7/29/41	2/3/42	510	-74.8	-5.98^{-6}	-.6431	-.5074	+.1357
2/3/42	7/30/42	687	+56.0	$+6.14^{-6}$	+.4944	+.3481	-.1463
7/30/42	2/19/43	891	-45.8	-6.06^{-6}	-.3991	-.2931	+.1060
2/19/43	8/5/43	1058	+31.6	$+6.18^{-6}$	+.2808	+.0334	-.2474
8/5/43	1/18/44	1224	-33.7	-6.19^{-6}	-.2999	-.1307	+.1692
1/18/44	7/23/44	1411	+43.1	$+6.11^{-6}$	+.3786	+.2524	-.1262
Totals to	7/23/44	1411	+29.2	$+6.12^{-6}$	+.2520	-.1100	±.1575
7/23/44	8/1/45	1785	+3.8	$+5.71^{-6}$	.0312	-.0450	-.0138
8/1/45	2/4/48	2702	-67.0	-6.25^{-6}	-.6021	-.5190	+.0831
2/4/48	8/19/48	2899	+72.8	$+6.20^{-6}$	+.6463	+.5280	-.1183
Totals to	8/19/48	2899	+38.5	$\pm 6.11^{-6}$	+.3274	-.1460	±.1318

(1) From adjustment curve above, based on Laboratory determinations of "e".
(2) Corrected for panel length: gage length ratio; from Column E, Table VI

perature, or $54^{\circ}+41.1^{\circ} = 95.1^{\circ}$F.

Similar computations for the data accumulated up to August 19, 1948 could be developed (with reservations due to the lack of complete seasonal measurements) using the average "e" and average seasonal moisture fluctuation data determined up to July 23, 1944. On this basis the total compensatory change in length would be equivalent to that which would be required for a temperature rise of 53.8°F. above hardening temperature or 107.8°F.

TABLE 8

CONTRACTION JOINT DEFORMATIONS

AVERAGE VERTICAL DEFORMATIONS OF EACH TYPE FROM SUMMER TO WINTER

(Inches in excess of 0. 05")
996 Joints Measured in 1944
502 Joints Measured in 1948

		NO METAL SEAL			STD. METAL SEAL			SP. METAL SEAL NO. 1			SP. METAL SEAL NO. 2			SP. METAL SEAL NO. 3		
ADJACENT PANEL LENGTH FT.	YEAR	ASPH. TOP SEAL	LATEX--OIL SEAL	R.A. TOP SEAL	ASPH. TOP SEAL	LATEX--OIL SEAL	R.A. TOP SEAL	ASPH. TOP SEAL	LATEX--OIL SEAL	R.A. TOP SEAL	ASPH. TOP SEAL	LATEX--OIL SEAL	R.A. TOP SEAL	ASPH. TOP SEAL	LATEX--OIL SEAL	R.A. TOP SEAL
15	1944	.085	.075	*	.058	.075	*	*	.050	-	.066	*	.050	.053	.150	*
	1948	.062	.075	*	.250	-	-	*	*	-	.067	*	*	.062	*	*
20	1944	.067	.070	.050	.082	.075	.050	.080	.100	*	.075	*	.050	.075	.088	.050
	1948	.050	*	-	.033	-	-	.100	*	*	*	*	-	.064	*	*
25	1944	.147	.114	.050	.150	.250	*	-	.100	-	.078	.100	.100	.125	.079	.125
	1948	.070	.100	-	-	*	-	-	-	-	.150	.100	-	.121	.175	.075
30	1944	.171	.150	.050	.194	.120	-	.325	.175	-	.160	.225	-	.154	.180	.225
	1948	.159	.075	*	.126	-	-	.150	.125	-	.150	*	-	.155	.183	.200
60	1944	-	.400	.275	.325	.375	.300	-	-	-	-	-	-	-	-	-
	1948	-	-	.450	.283	.375	.250	-	-	-	-	-	-	-	-	-

* Less than 0.05"

These increases in expansion space due to shrinkage and the compensatory effect of seasonal moisture changes are both in addition to the original expansion space built into the pavement. Neglecting any reduction in this space due to foreign material infiltrating the joint openings, this would theoretically mean that the original expansion joints would not be required to function as such at temperatures below 95. 1°F. after 4 years or below 107. 8°F. after 8 years of service.

That infiltrated material may become a serious matter, where the pavement design places no restraint on the expansion and contraction of individual panels, is ap-

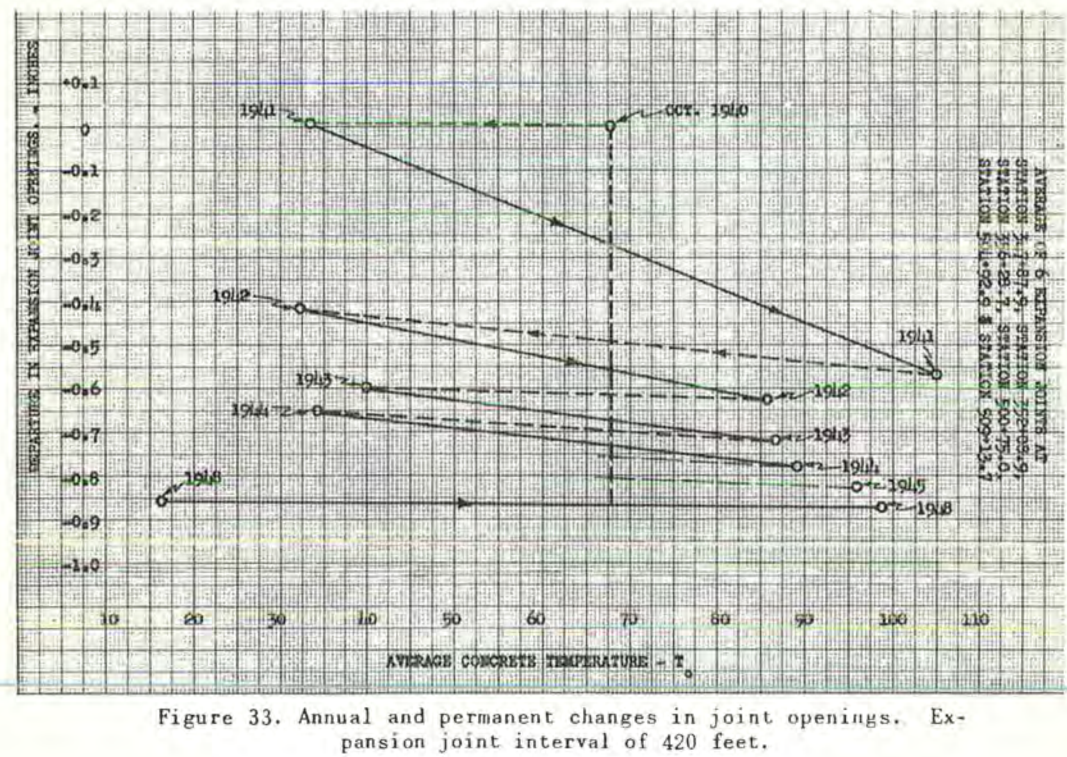


Figure 33. Annual and permanent changes in joint openings. Expansion joint interval of 420 feet.

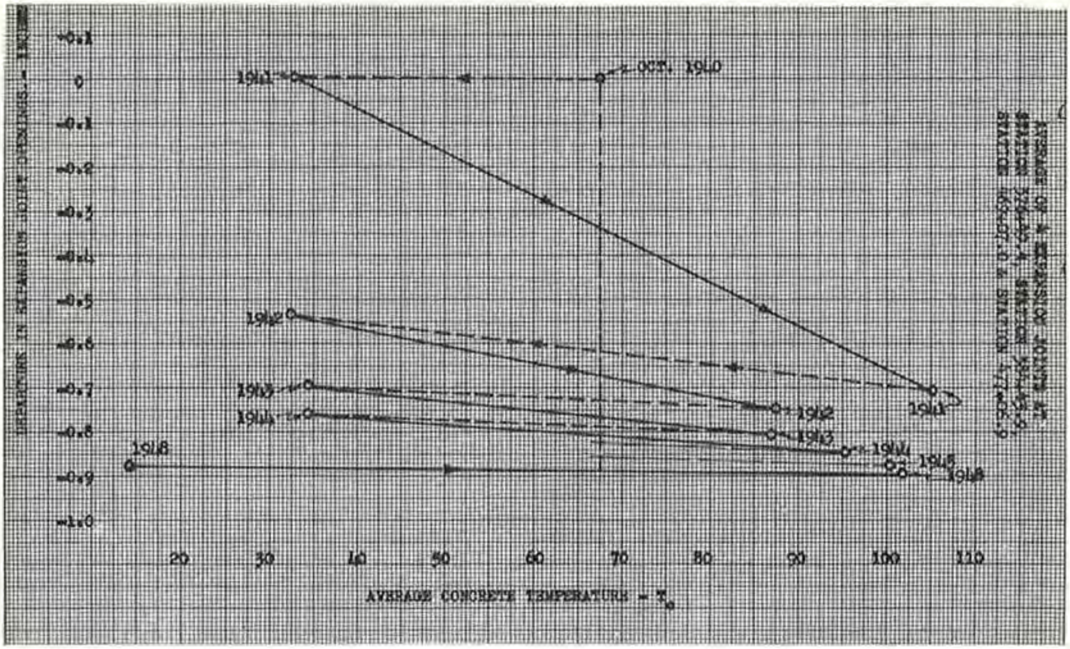


Figure 34. Annual and permanent changes in joint openings. Expansion joint interval of 800 feet.

parent from inspection of Figure 40. This figure shows the progressive change in opening of both the expansion and contraction joints on this particular section (Sta. 524+95. 5 to Sta. 526+15. 4) with reference to the initial condition. As built, all of the expansion space (one inch) was concentrated in the expansion joint since there were no openings at the dummy type contraction joints. The change in distribution

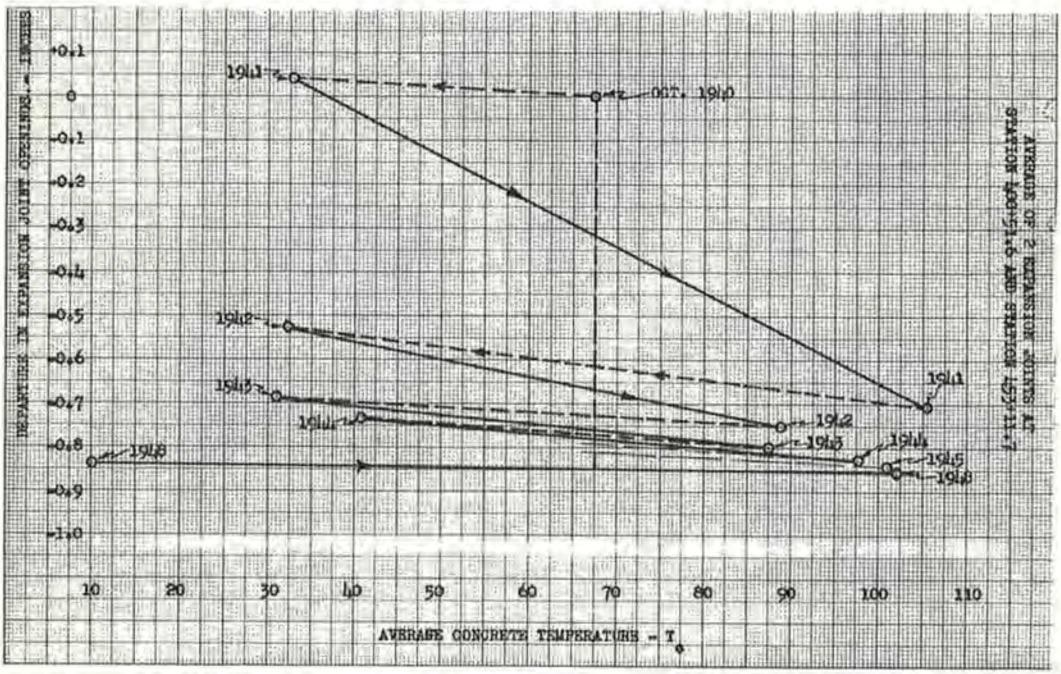


Figure 35. Annual and permanent changes in joint openings. Expansion joints at ends of division 9.

of this space and the migration of the panels toward the expansion joints are clearly shown. On July 23, 1944 the space in the expansion joint was reduced from one inch to 0.5317 inch and by August 18, 1948 to 0.2499 inch. Meanwhile the contraction joints had accumulated a total opening of 0.5783 inch by July, 1944 and 0.8961 inch by August, 1948. It is interesting to note that the sum of the openings on these dates was 1.1100 inch in July, 1944 and 1.1460 inch in August, 1948; 0.1100 inch and 0.1460 inch greater than the expansion space originally built into this section.

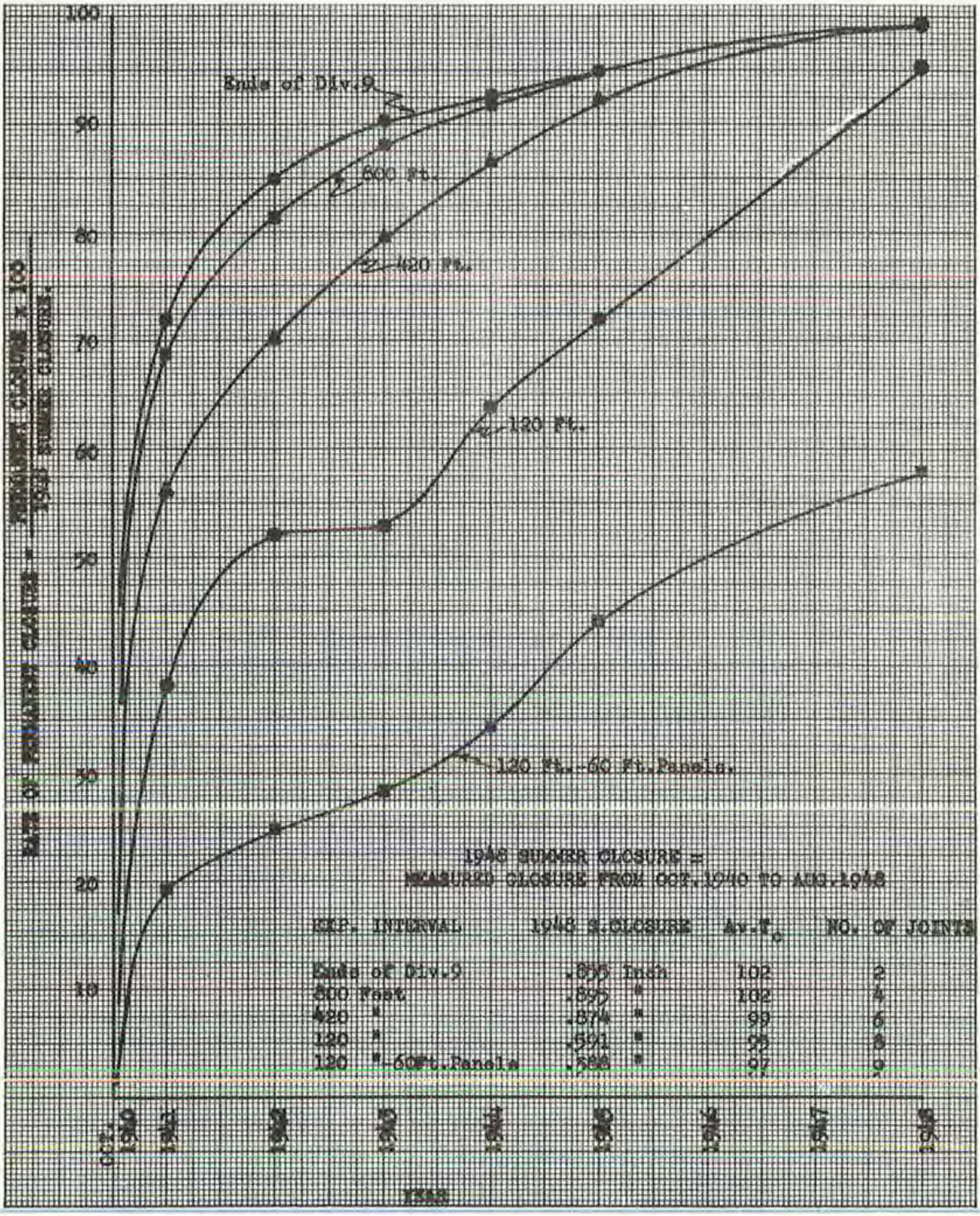


Figure 36. Rate of permanent closure of expansion joints for various expansion intervals.

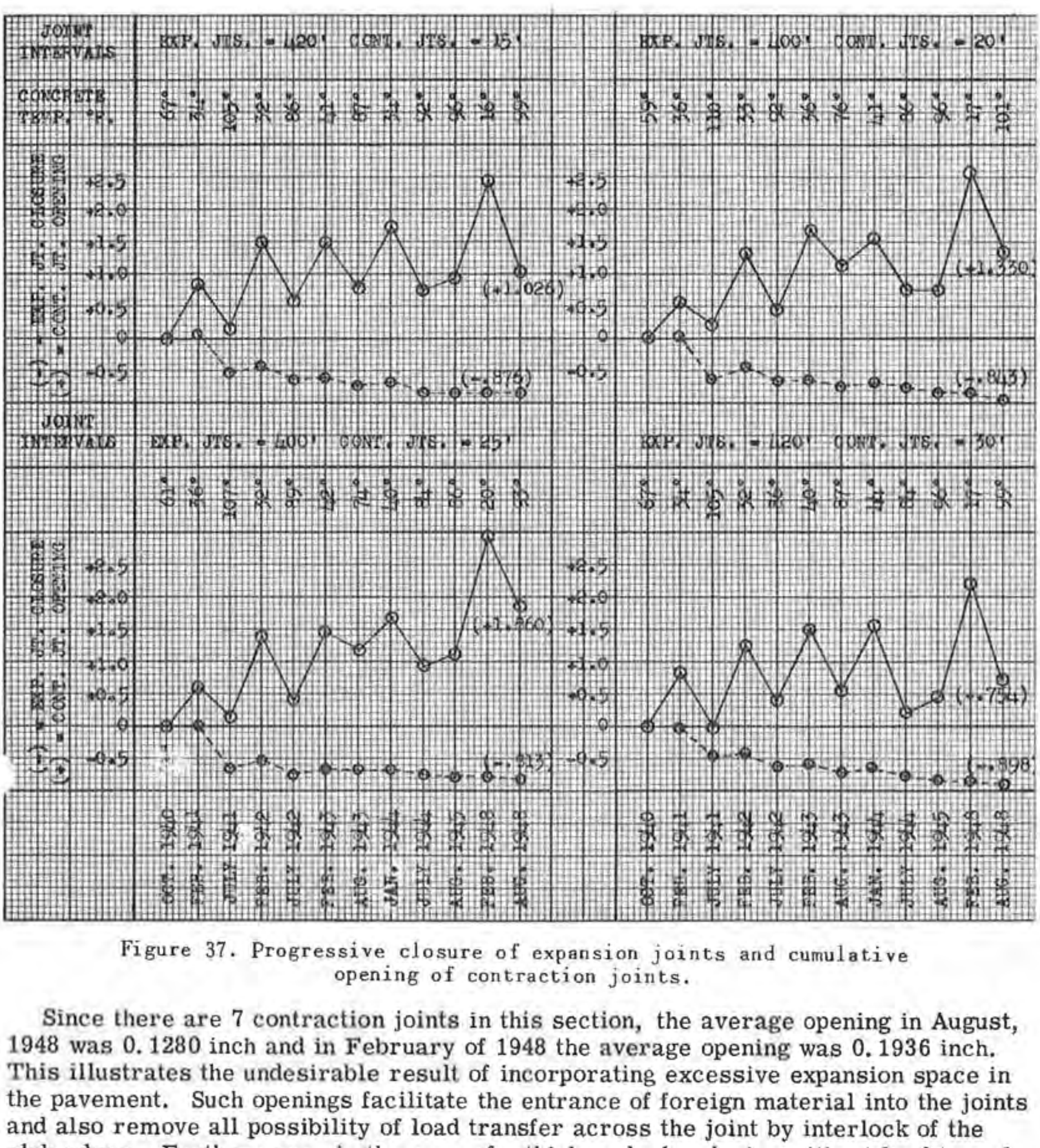


Figure 37. Progressive closure of expansion joints and cumulative opening of contraction joints.

Since there are 7 contraction joints in this section, the average opening in August, 1948 was 0.1280 inch and in February of 1948 the average opening was 0.1936 inch. This illustrates the undesirable result of incorporating excessive expansion space in the pavement. Such openings facilitate the entrance of foreign material into the joints and also remove all possibility of load transfer across the joint by interlock of the slab edges. Furthermore, in the case of a thickened edge design without load transfer,

TABLE 9

1950 CRACK & ROUGHOMETER SURVEY

Cracking & Roughness Index as Related to Panel Length

Panel Length	Total No. Panels	Total No. Panels Cracked Trans.	Total No. Panels Cracked Long.	Percent of Total Panels Cracked Trans-versely	Percent of Total Panels Cracked Longi-tudinally	Average Spacing of Trans. Opening	No. Trans-verse Cracks Per Mile	Average No. of Trans. Openings Per Mile	Rough-ness Index In. Mile
15'	1239	15	30	1.2	2.4	14.8	4	357	106.7
20'	928	31	70	3.3	7.5	19.4	9	272	104.2
25'	604	80	43	13.2	7.1	22.1	28	239	104.0
30'	624	138	81	22.1	12.98	24.6	39	215	105.3
30' Reinforced	56	36	3	64.3[a]	5.4	18.3	113	289	91.5
30' Reinf. W/15' Cracker Strip	121	0	6	0.0	5.0	15.0	–	352	103.5
60' Reinforced	96	17	5	17.7	5.2	51.0	16	104	95.2

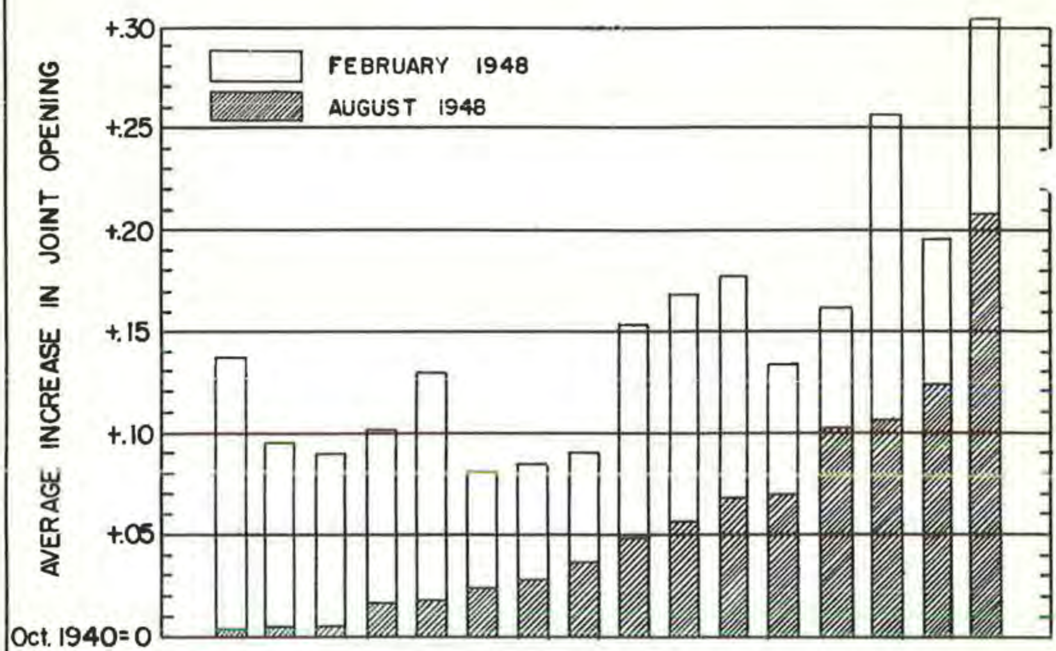
[a] Possibly due to subgrade condition.

a free edge is created in the pavement where the section is the weakest.

LONGITUDINAL COMPRESSIVE STRESSES

During construction special extensometer points were installed at various selected locations throughout the project for use in taking measurements which, it was thought, would be of value in determining a close approximation of longitudinal stresses due to temperature changes. In general, these special points were set in the middle of a panel which was located at, or very close to, the mid-point between expansion joints. At each installation a series of seven points were set in a row longitudinally with the pavement and with a spacing between points of approximately ten inches. Thus a series of points covered a pavement length of approximately 60 inches. A special extensometer,

CONT. JOINT INTERVAL	30	20	25	20	25	15	15	15	20	30	30	20	15	30	25	60
EXP. JOINT INTERVAL	5260	800	800	5260	5260	5260	795	420	120	420	810	400	120	120	400	120



AVERAGE CHANGE IN JOINT OPENINGS (Inches)																	
	AUG. 1948	0.004	0.005	0.005	0.016	0.017	0.025	0.029	0.038	0.049	0.058	0.068	0.070	0.102	0.108	0.124	0.208
	FEB. 1948	0.139	0.096	0.091	0.101	0.129	0.081	0.085	0.091	0.154	0.170	0.178	0.135	0.161	0.258	0.196	0.309

Figure 38. Average increase in contraction joint openings.

TABLE 10

1950 CRACK SURVEY

Longitudinal Cracking as Related to Panel Length and Pavement Section

Panel Length	Section	Total No. Panels	Total, Ft. Longitudinal Cracks	Ft. of Crack Per Panel (Average)	Ft. of Crack Per Mile (Average)
Feet	Inches				
15	9-6-9	1047	258	.246	173
15	7	192	30	.156	110
20	9-6-9	788	782	.992	524
20	7	144	23	.160	84
25	9-6-9	484	413	.853	360
25	7	120	14	.117	49
30	9-6-9	528	979	1.854	653
30	7	96	0	0	0
30 (Reinforced)	9-6-9	56	103	1.839	647
30 (Reinf. 15 Crackerstrip)	9-6-9	121	37	.308	108
60 (Reinforced)	9-6-9	96	46	.479	84

TABLE 11

1950 CRACK SURVEY

Cracking as Related to Pavement Section for Various Panel Lengths

Panel Length	Section	Total No. Panels	No. of Trans. Cracked Panels	No. of Trans. Cracks Per Mile	Ft. of Longitudinal Cracks	Feet Longitudinal Cracks Per Mile
Feet	Inches					
15	7	192	5	9.2	30	110.0
15	9-6-9	192	0	0	88	322.7
20	7	144	3	5.5	23	84.3
20	9-6-9	144	1	1.8	34	124.7
25	7	120	1	1.8	14	49.3
25	9-6-9	120	17	29.9	18	63.4
30	7	96	22	40.3	0	0
30	9-6-9	96	23	42.1	149	546.3
Total & Av.	7				67	60.8
Total & Av.	9-6-9				289	262.2

Note: Data from Divisions 2, 3, 15 & 16 for 7 inch uniform depth and Divisions 5, 6, 12 & 13 for 9-6-9 pavement section. Only variable in this comparison is pavement cross section. 120 and 125 foot Expansion Joint Intervals.

reading to 0.0001 of an inch, was used in making measurements and the operation of this instrument was controlled at all times by reference to a standard "Invar" bar. Figure 41 shows some of the instrumentation details relating to the arrangement of these points and thermocouple installations for temperature control.

Figures 42, 43 and 44 show data secured by these measurements and stress computations for the summers of 1944, 1945 and 1948 at four points on this project.

The linear relationship, No. 1 in the figures, shows the unit change in length of the concrete over a range of slab temperatures from 72°F. to 105°F. The installation from which these data later were obtained was located as close to an expansion joint as it was possible to place the seven points; that is, the length measured was the 60 inches immediately adjacent to the expansion joint. This joint was one of a long series which were spaced at 120-foot intervals. This location was used as a point of reference in the stress determinations and was selected because restraint against free expansion

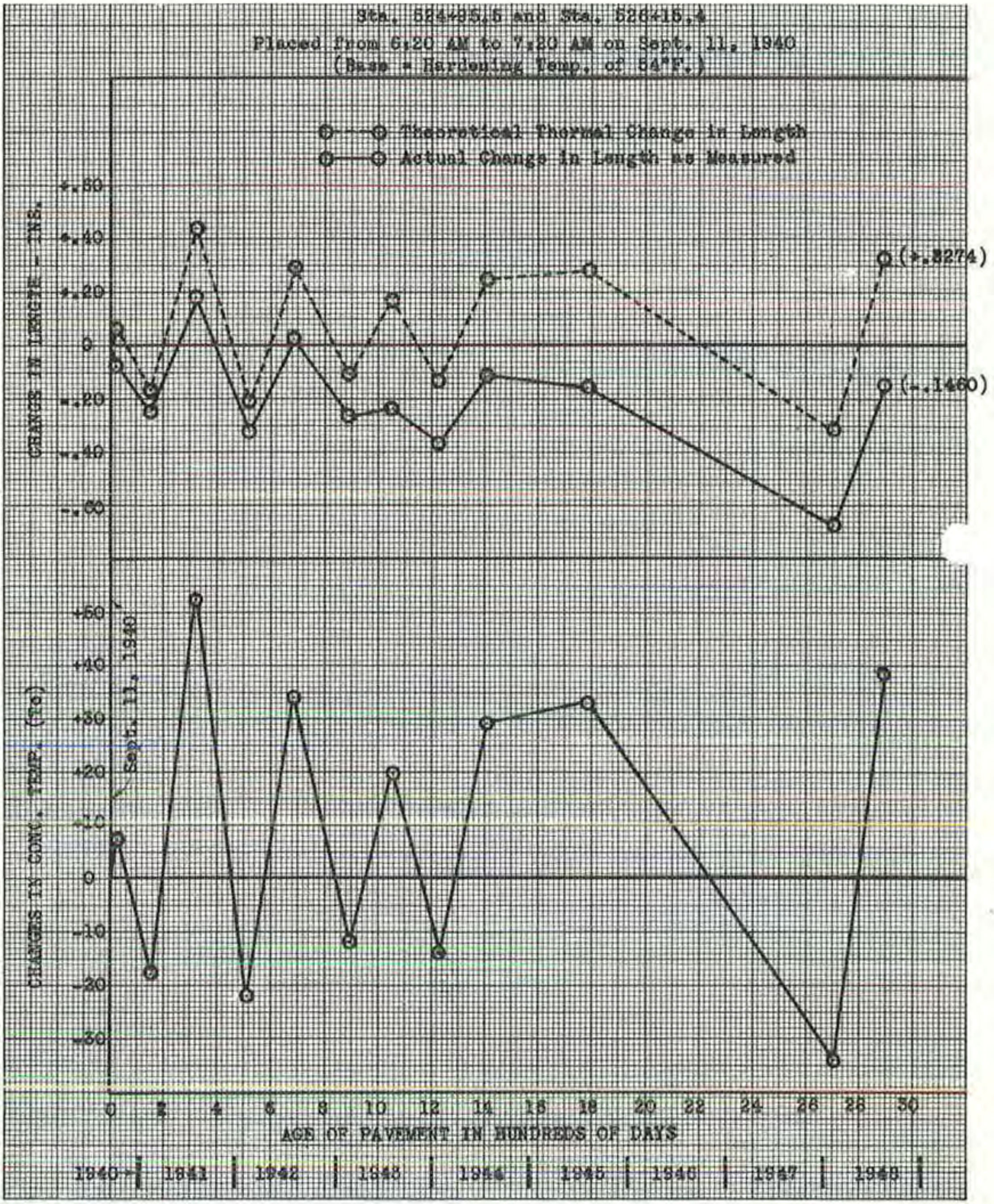


Figure 39. Annual changes in concrete temperature and length of 120 ft. of pavement between expansion joints.

TABLE 12

1950 CRACK SURVEY

Cracking as Related to Panel Length in Restrained Concrete

Effect of panel lengths in long sections of restrained concrete pavement (Exp. Jt. Intervals of 400 Ft. to 1 Mile) with respect to longitudinal and transverse cracking.

9-6-9 Section

Panel Length	Total No. Panels	No. of Trans. Cracked Panels	No. of Trans. Cracks per Mile	Ft. of Long. Cracks	Feet Long. Cracks Per Mile
15	855	10	4.1	170	140
20	644	27	11.0	748	613
25	364	62	36.0	395	458
30	432	93	37.9	830	676
				Average	472

was at a minimum. The thermal coefficients of the concrete, as obtained by the field measurements on the various dates, were 1944, e = .0000053; 1945, e = .00000499 and 1948, e = .00000457. The laboratory determination was .00000545 for this concrete in the corresponding temperature range shortly after construction. There appears to be a gradual reduction in the thermal coefficient as indicated by the field measurements.

The curves numbered 2 show the unit change in length at the midpoint of Division 9 which has a length of 5,280 feet and contains no expansion joints. At the lower end of the temperature range, these curves tend to parallel the straight-line curve No. 1; but, at the higher temperatures they curve to the right and become horizontal indicating full restraint with no further expansion though the temperature continues to rise.

Curves No. 3 and 4 show the unit change in length at the quarter points of Division 9 which are approximately ¼ mile from the expansion joints. Both of these curves show restraint characteristics similar to Curve No. 2, indicating that full restraint is built up within something less than ¼ mile from the expansion joints.

In 1944 the temperature at which restraint occurred was not as well defined as in the subsequent years, there being a transition from free expansion to full restraint through temperatures of 75±°F. to 95±°F. In 1945 the transition from free expansion to full restraint occurred in a much narrower range of temperatures, 78°F. to 82°F. Again in 1948 the temperature range was small, 77.5°F. to 82.5°F. and oddly Curves No. 2 and No. 3 plotted as a single curve. Generally the data indicates that the slab at the ¼ points reached full restraint at a slightly lower temperature than at the midpoints.

TABLE 13

1950 CRACK SURVEY

Effectiveness of Dowels in Preventing Faulting

		Doweled Joints						Joints Not Doweled					
		7" Section			9-6-9 Section			7" Section			9-6-9 Section		
Exp. Jt. Interval	Contraction Jt. Interval	Total Jts.	Fltd. Jts.	% Fltd. Jts.	Total Jts.	Fltd. Jts.	% Fltd. Jts.	Total Jts.	Fltd. Jts.	% Fltd. Jts.	Total Jts.	Fltd. Jts.	% Fltd. Jts.
Feet	Feet												
120	15	50	--	0	50	1	2.0	50	16	32.0	50	3	6.0
	20	32	2	6.3	38	--	0	38	5	13.2	38	3	7.9
	25	32	--	0	32	--	0	32	8	25.0	32	8	25.0
	30	26	2	7.7	25	--	0	26	8	30.8	26	8	30.8
	30 Reinf.	0			0			0			0		
	60 Reinf.	0			46	1	2.2	0			0		
400+	15				0						436	28	6.4
	20				97	2	2.1				234	19	8.1
	25				0						190	42	22.1
	30				64	3	4.7				162	32	19.8
	30 Reinf.				90	2	2.2				0		
	60 Reinf.				5	2	40.0				0		

15 Faulted Doweled Jts. out of 587 Jts. or 2.6%.

180 Faulted Jts. out of a total of 1,314 Jts. or 13.7%.

Fltd. - abbreviation for Faulted.

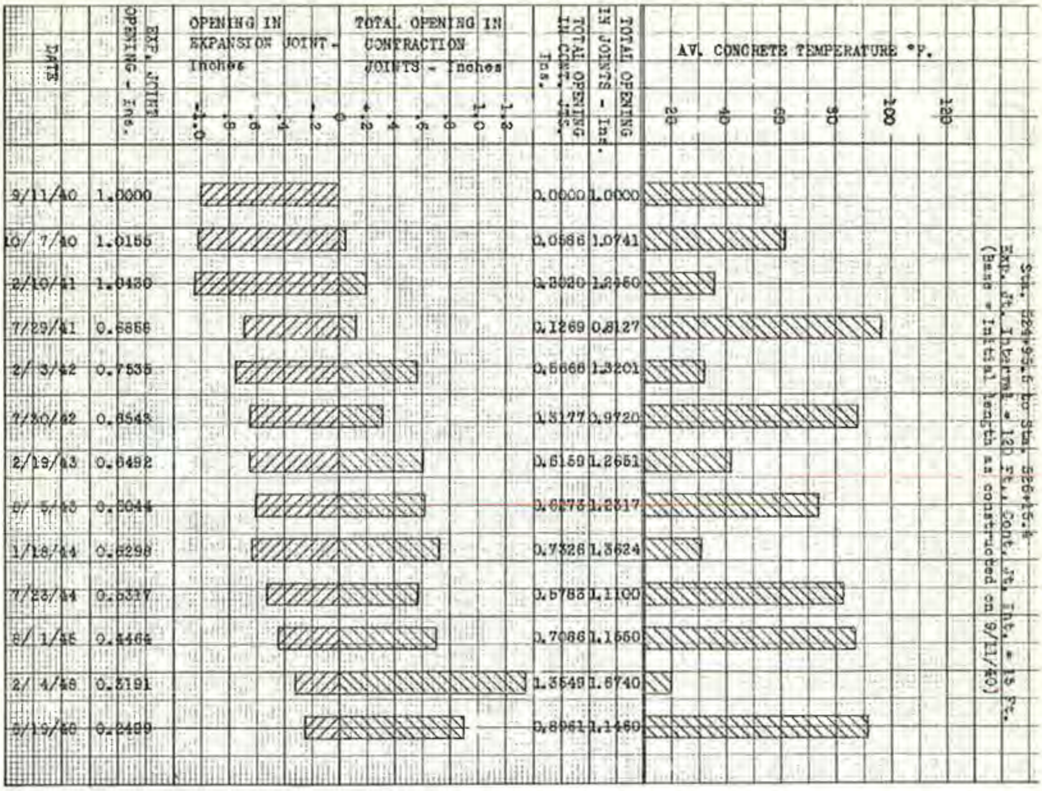


Figure 40. Progressive Closure of expansion joints and cumulative opening of contraction joints.

These curves may be used to compute the approximate compressive stress that existed at these locations on the days when measurements were made. It should be kept in mind that the computed stresses apply only to these points and to the dates given. They should not be construed as applying generally to other locations and dates.

The highest average slab temperature found on this project up to the summer of 1944 was 112°F. A close approximation of the longitudinal stress caused by this temperature can be determined from the curves as follows:

(1) Select some temperature value below the point of tangency of curves 2, 3 and 4, say 70°F. and read the unit change in length (X) for all curves. Similarily read the unit change in length (Y) at 112°F. The numerical sum of the X and Y readings is the total unit change in length from 70°F. to 112°F.

(2) For Curve No. 1, X + Y represents the unit change in length associated with free expansion, indicated as K; therefore, when the X + Y values for Curves 2, 3 and 4 are

TABLE 14

1950 CRACK SURVEY

Effect of Panel Length and Expansion Interval on Faulting of 9-6-9 Sections without dowels in Division 5 thru 13.

	EXPANSION INTERVAL											
	120 to 125 ft.			400 to 420 ft.			795 to 810 ft.			5260 ft.		
Length of Panel	No. Panels	Faulted Joints	Per-cent	No. Panels	Faulted Joints	Per-cent	No. Panels	Faulted Joints	Per-cent	No. Panels	Faulted Joints	Per-cent
15	96	3	3.1	224	8	3.6	208	7	3.4	176	1	0.6
20	72	3	4.2	160	7	4.4	160	6	3.8	132	6	4.5
25	60	8	13.3	128	26	20.3	128	13	10.2	104	3	2.9
30	48	8	16.7	112	15	13.4	108	7	6.5	88	10	11.4
Total & Av.	276	22	8.0	624	56	9.0	604	33	5.5	500	20	4.0

704 panels of 15 foot length are shown in this tabulation with 19 faulted joints or 2.7%.

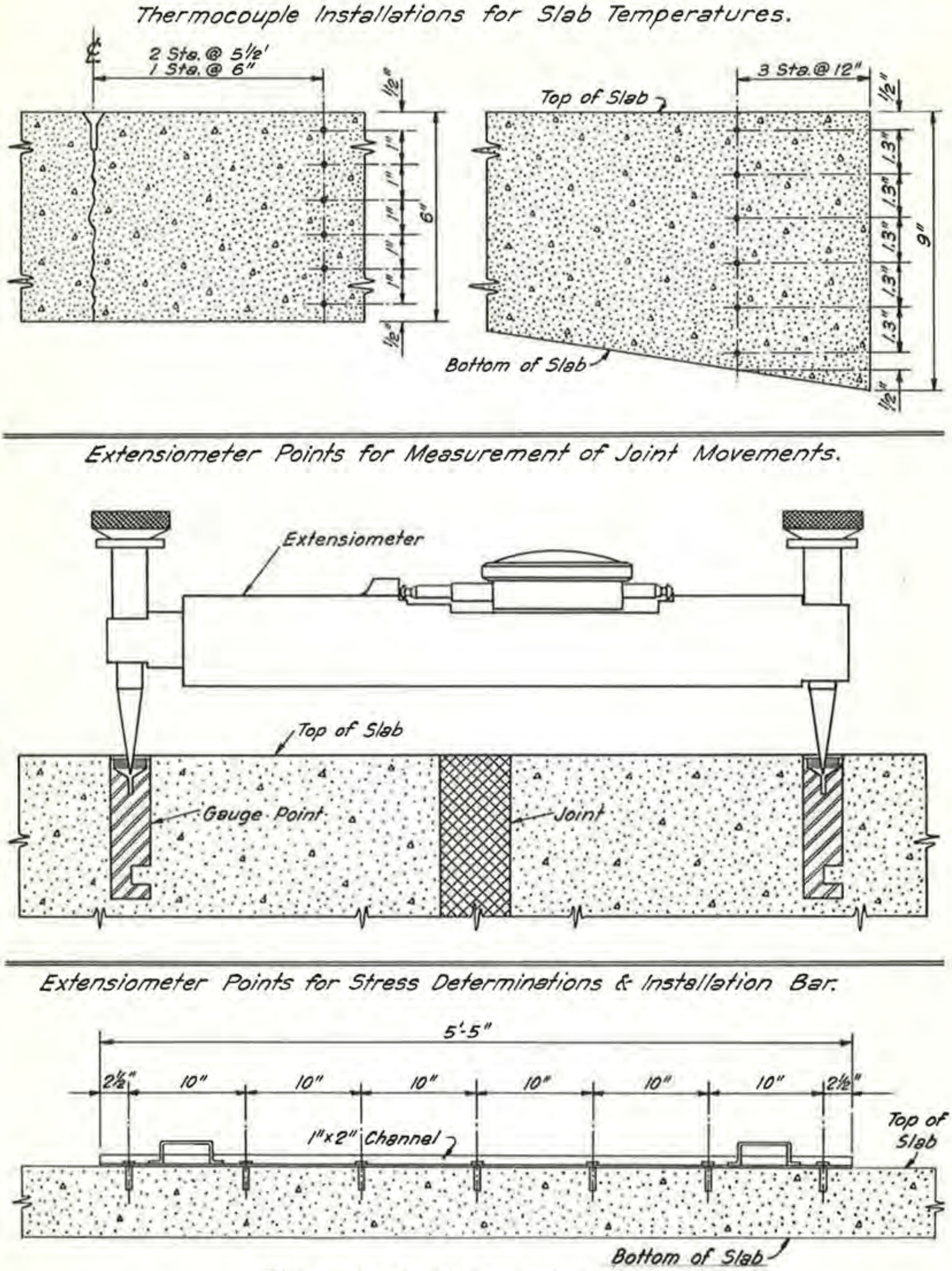


Figure 41. Instrumentation details.

subtracted from K, the difference represents the amount of restrained expansion, K_1.

(3) The unit stress may then be computed from the stress-deformation relationship if the modulus of elasticity is known. According to laboratory tests on this concrete, the average E for ten determinations was 4, 300, 000; therefore, unit stress = 4, 300, 000 x K_1 x 10^{-5}.

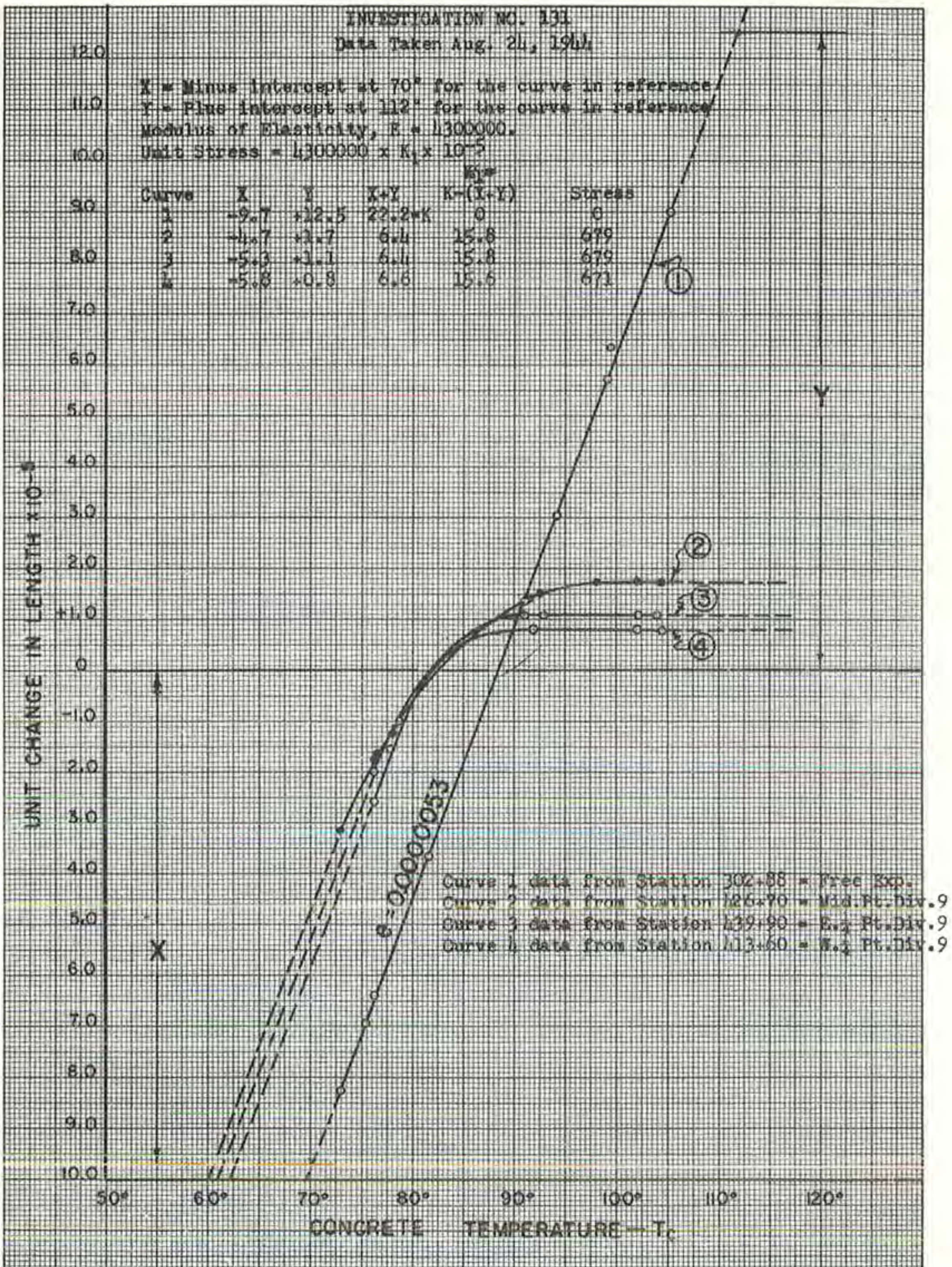


Figure 42. Unit stress determinations.

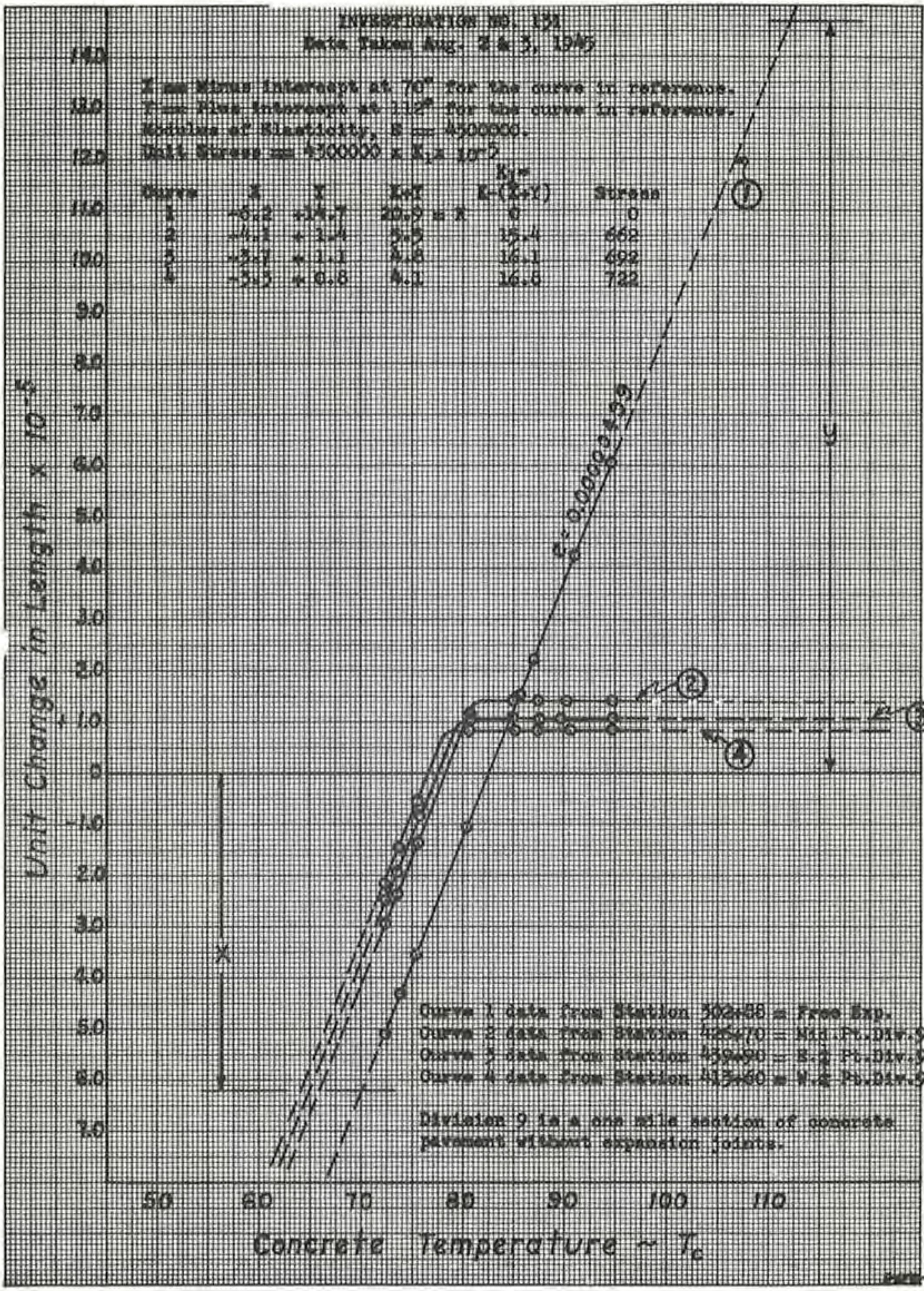


Figure 43. Unit stress determinations.

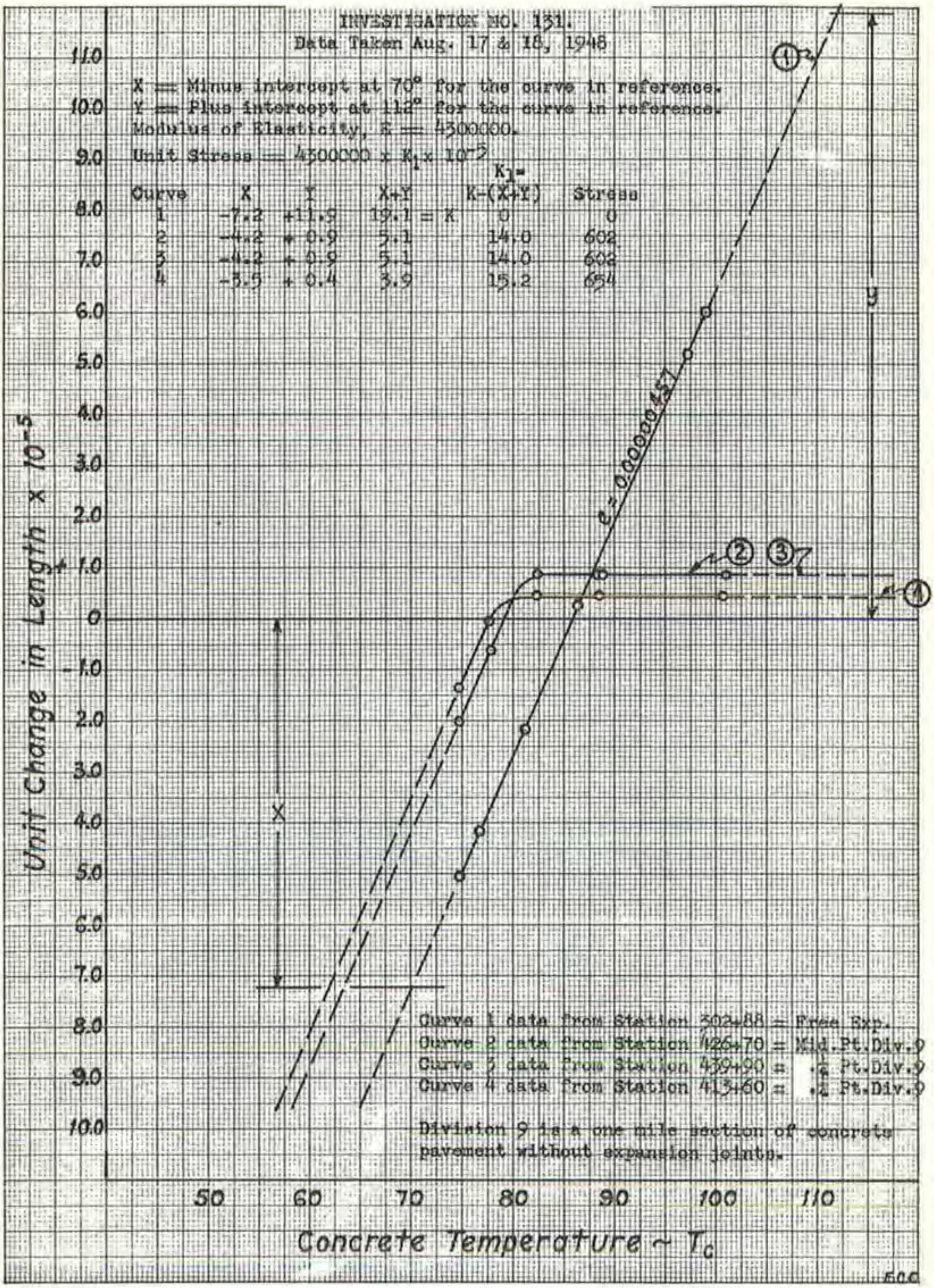


Figure 44. Unit stress determinations.

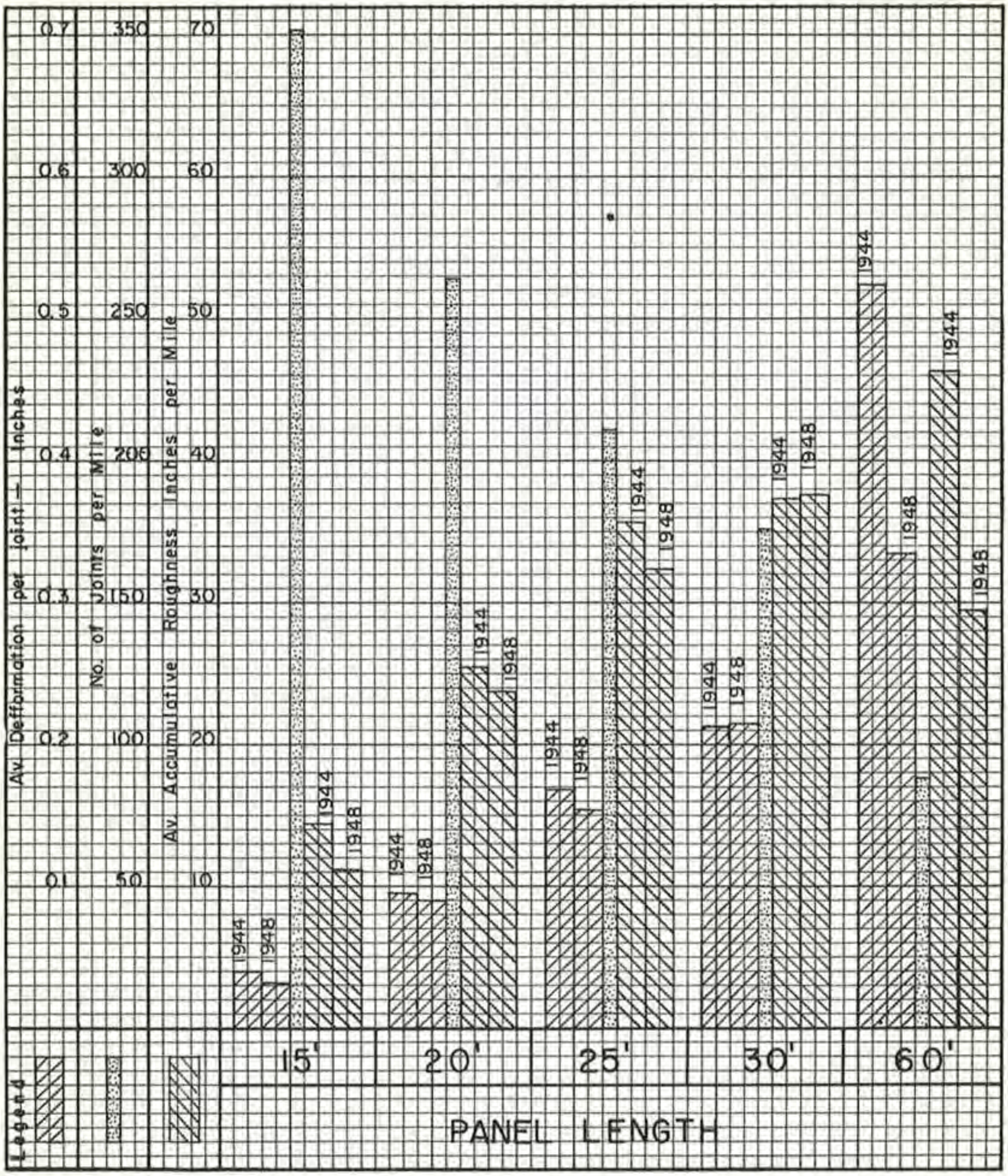


Figure 45. Effect of panel length on roughness.

The computed unit stress values on the various dates were as follows:

	Center Div. 9	East ¼ Point	West ¼ Point
August 24, 1944	679	679	671
August 2 and 3, 1945	662	692	722
August 17 and 18, 1948	602	602	654

Since these computations were based on measurements made directly on the pavement during a 24-hour period and at two different points on the project, one of which permitted free expansion and the other being under restraint, it is felt that they provide a close approximation of the actual stresses which would exist in the pavement at

TABLE 15

1950 CRACK SURVEY

Relation of Spalled Joints with Expansion and Contraction Joint Interval

	120 Ft. Exp. Int.						400 + Ft. Exp. Int.		
Cont. Jt. Int. Ft.	9-6-9 Section	Total Number	Per-cent	7" Section	Total Number	Per-cent	9-6-9 Section	Total Number	Per-cent
15	0	100	0	6	100	6	17	436	3.9
20	5	76	6.5	2	70	2.9	7	331	2.1
25	2	64	3.1	4	64	6.2	8	190	4.2
30	1	51	1.9	7	52	11.1	8	226	3.5
30R	0	0	0	0	0	0	11	90	12.2
60'R	3	46	6.5		0	0	0	5	0
Total & Av.	11	337	3.3	19	286	6.6	51	1278	4.0

these times at a temperature of 112°F. The effect of moisture content of the concrete was a minimum since there had been no precipitation during the 24-hour periods of measurement nor had precipitation occurred for a considerable time prior to these periods.

These determinations indicate that no serious compressive stresses have developed in this pavement up to 8 years after construction. The indicated stresses are about only $\frac{1}{7}$ the ultimate compressive stress of this concrete.

JOINT DESIGNS AND MATERIALS

Expansion Joints

The expansion joints on this project were all one inch in width and intervals between joints ranged from 120 feet to one mile. Variables in design consisted of the use of copper seals in some joints, while in others this seal was omitted; three different filler or core materials, cane and wood fiber premolded materials and ground cork and asphalt poured type; four different top sealing materials consisting of asphalt-diatomaceous earth mixture, a latex-oil mixture, a manufactured rubber material (Rubber Associates) and premolded rubber strips manufactured by the Goodrich Rubber Company. In addition to the above variables, some joints did not include dowels or other load transfer devices. The various combinations resulted in a total of 30 different joint designs.

In 1944, four years after construction, a total of 102 expansion joints were checked for vertical deformation by string measurements; of these, 62 contained copper seals and 40 did not. The string measurements were read to the closest 0.05 inch and joints which showed deformations of 0.05 inch or less between summer and winter were considered as not having changed. These data indicated little, if any, reduction in deformation from the use of copper seals when considering only those joints with deformations greater than 0.05 inch. However, considering all the joints measured, those that contained copper seals showed 41.7 percent of their number having deformations greater than 0.05 inch while those having no copper seals showed 62.5 percent having deformations in excess of 0.05 inch. In the case of joints located between 60-foot reinforced panels, some of the joints which contained copper seals showed deformations as great as, or greater than, those without such seals.

This may be an indication that, in general and where panel lengths are not excessively long, copper seals may for a time be somewhat beneficial in reducing the magnitude of seasonal deformations during the early life of the pavement. However, there was no positive indication that their use prevented the development of these deformations, especially in view of the progressive closure and reduced seasonal movement of the expansion joints.

Based on data obtained up to 1944 on the effectiveness of top-sealing materials, it was concluded that there was little difference between asphalt-diatomaceous earth, latex-oil and premolded rubber strips. All of these indicated several times as much joint deformation as the Rubber Associates material. Examination of the expansion joints in 1950 showed that the closure of the joints had resulted in the general extrusion of the filler and top-sealing materials to the extent that it was impossible to determine their effectiveness.

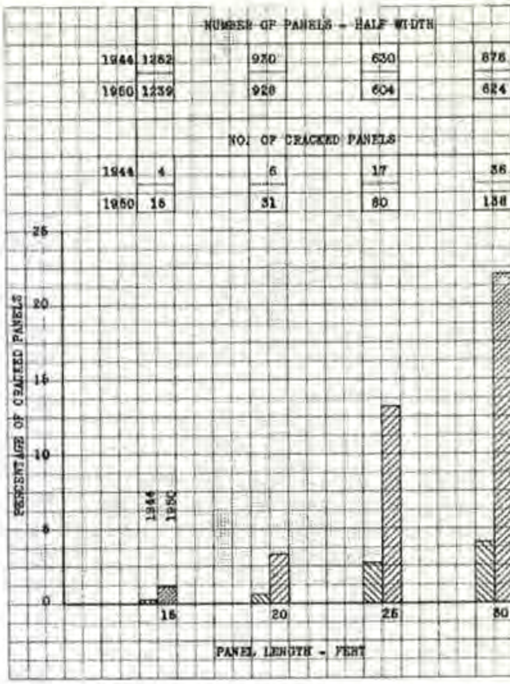


Figure 46. Relationship of transverse cracking to panel length.

Contraction Joints

Dummy type contraction joints were used exclusively on this project. However, a total of 18 different designs were used; the variations in design being due to the use of various types of metal seals, the use of asphaltic, latex-oil and rubber top seals and the use of dowels in some cases and their omission in others.

String measurements were made on 996 contraction joints in the summer and winter of 1944 and on 502 joints in 1948. The average joint deformations for various design features are shown in Table 8. It is interesting to note that the larger deformations are associated with the longer lanel lengths. These data are further summarized to show the effect of the principal variables as follows:

Copper Seals. A point by point analysis covering all panel lengths and using the joints having no metal seals as a basis indicated the following:

	1944	Percent	1948	Percent
Deformations reduced	17 cases =	39	13 cases =	43
No reduction	27 cases =	61	17 cases =	57
	44 cases =	100	30 cases =	100

Latex-Oil Seal. An analysis similar to that used for metal seals, except that the asphaltic material is used as a basis, indicated:

	1944	Percent	1948	Percent
Deformations reduced	9 cases =	45	9 cases =	56
No reduction	11 cases =	55	7 cases =	44
	20 cases =	100	16 cases =	100

Rubber Associates Material. Compared to the standard asphaltic material, this material showed the following performance:

	1944	Percent	1948	Percent
Deformations reduced	13 cases =	81	8 cases =	89
No reduction	3 cases =	19	1 case =	11
	16 cases =	100	9 cases =	100

The above analysis indicates a superiority of the Rubber Associates material over the other top-sealing materials. However, in 1950, a survey of 31 contraction joints sealed with Rubber Associates material showed that only 7 were not open and rated as being in fair to good condition. The remaining 24 joints were open from $\frac{1}{16}$ to $\frac{1}{4}$ inch and generally in poor condition. These joints were located in the various divisions of the project and associated with various panel lengths. However, the maintenance of these joints was omitted, except where positively necessary, throughout this 10-year period.

PRESENT CONDITION OF THE PAVEMENT

General

The general condition of this pavement was fairly good in 1950, ten years after construction. It has shown a moderate increase in roughness and an increased rate of transverse and longitudinal cracking with age. Spalling of the concrete at joints has not been extensive; although a considerable number of cracks have spalled, probably due to a lack of maintenance. There has been one blow-up and one partial failure due to compressive stresses. Faulting of joints and cracks has been moderate both as to number and magnitude. Dowels have been effective in reducing faulting at joints. The 7-inch uniform pavement section appears to be slightly superior to the 9-6-9 section, but neither was entirely adequate as indicated by longitudinal cracking and corner breaks.

Changes in Smoothness

Vertical deformations in the pavement surface, due to the development of differentials in elevation at the transverse joints with reference to the elevation of the midpoint of the adjacent panels, first developed on this project during February of 1943. At that time the high joints were confined to the 2,000 feet at the extreme southwest end of the project and the magnitude of the deformations was very slight. The following winter, in January of 1944, high joints were of greater magnitude and were noticeable over most of the project. In the winter of 1948 measurements indicated that the joint deformations were much the same as in 1944. The data for these two years are shown in Figure 45 along with the accumulated roughness in inches per mile for the various panel lengths. The increase in average joint deformation and accumulated roughness per mile for increasing panel lengths is quite evident from this figure.

The above vertical deformations were obtained by string measurements. A strong silk fishing line was stretched across the joints with the ends supported at the mid points of the adjacent panels and three inches above the pavement surface. The line was maintained under a constant tension of ten pounds and measurements to the nearest 0.05 inch were made from the pavement surface at the joint upwards to the line. The readings were corrected for sag and gradient curvature as required. In 1944 measurements were made on 1,100 joints; whereas, in 1948 only 534 joints were measured.

Precise level points were installed during construction for subsequent use in determining changes in smoothness. Unfortunately these points were damaged the following winter by ice-removal operations. However, repeated checks have been made with the roughness recorder, which is a duplicate of the machine described in Volume 20 of the Proceedings of the 20th Annual Meeting of the Highway Research Board. (3) The average roughness values obtained with this machine were as follows:

Date	Inches per Mile
Nov. 1941	85
Feb. 1942	84
July 1944	96
Nov. 1949	100
Nov. 1950	100.5

These values, while expressed in inches per mile, should not be construed as being absolute values in those terms; they merely represent the accumulation, in inches, of the spring deflections of the machine as influenced by the pavement roughness. The values are significant only in making comparisons of the relative roughness of different pavements or of the same pavement at different times.

It is apparent that only a moderate increase in roughness has occurred during the ten year period. The right-hand column of Table 9 indicates how this roughness was associated with various panel lengths in 1950.

In comparison with data secured with this recorder on a considerable mileage of other pavements, varying in roughness from good to bad, this pavement would rate as better than average.

Transverse Cracking:

Transverse cracking has progressed with time. The first year there was very little. By 1944 there were 110 such cracks of which 47 were accounted for by evidence of subgrade subsidence or frost action leaving 63 or 57 percent attributed to temperature. None of these were in reinforced panels. The 1950 crack survey disclosed 317 panels cracked transversely due to temperature changes. For these years, the relationship of transverse cracking to panel length is shown in Figure 46 and Table 9. It is apparent that the short panels have quite effectively controlled cracking, there being only 1.2 percent of the 15-foot panels cracked whereas the 30-foot panels were 22.1 percent cracked in 1950.

Additional analysis of transverse cracking in 1950 is shown in Table 11 and 12. These data indicate that, for 15 and 20-foot panels, transverse cracking was less on

TABLE 16

1950 CRACK SURVEY

Corner Cracks

Design	Expansion Joint Interval	Internal Corners Constr. Joint	Internal Corners Exp. Joint	Internal Corners Contr. Joint	External Corners Const. Joint	External Corners Exp. Joint	External Corners Contr. Joint	Total Cracks	Total Panels	Percent Cracked
9-6-9	120 Ft.	--	1	2	0	2	3	8	640	1.25
9-6-9	400 Ft. & over	1	2	22	-	-	21	46	2,474	1.9
7"	120 Ft.	-	-	2	-	1	5	8	552	1.5

the 9-6-9 pavement section than on the 7-inch uniform section, with the reverse being true for the 25 and 30-foot panels. On long sections of restrained concrete the relationship of transverse cracking to panel length was in the same order as the overall average; however, the percentage of cracked panels was greater for all panel lengths, indicating that the concrete pavement in restraint cracked transversely to a greater degree than that not in restraint.

Over the entire project, 7 percent of the transverse cracks due to all causes had faulted in 1950.

Longitudinal Cracking

In 1944 a total of 405 linear feet of longitudinal cracking had occurred. All of this, except one 30-foot crack, was confined to Division 1 on the southwest end of the project, and was approximately at the right ¼ point. In 1949 the cracking in Division 1 had increased to 732 feet and the total for the project was 1,759 feet. By 1950 the total for the project was 2,685 feet, an increase of 52 percent in that year, indicating the rapid progression of longitudinal cracking during the tenth year.

Longitudinal cracking in relation to panel lengths is shown in Table 9 for the entire project. For non-reinforced panels, the percent of panels cracked longitudinally was least for 15-foot panels (2.4 percent), about equal for the 20 and 25-foot panels (7 percent) and greatest for 30-foot panels (13 percent). The 30 and 60-foot reinforced panels showed about an equal percentage of panels cracked (5 percent) which was more than twice the amount for the 15-foot non-reinforced panels.

A better comparison is shown in Table 10 where the length of longitudinal cracking is shown for the various panel lengths. On this basis, considering only the 9-6-9 sections, the 15-foot panels show the least cracking, 173 feet per mile, the 25-foot panels 360 feet per mile, the 20-foot panels 524 feet per mile and the 30 foot panels the most at 653 feet per mile. This is the same order as indicated in Table 9. The reinforced panels, however, show a wider variation than previously indicated, the 30-foot panels averaging 647 feet per mile as compared to 84 feet per mile for the 60-foot panels and the 30 foot panels with 15-foot cracker strip having 108 feet per mile; whereas, in Table 9 the percent of cracked panels was nearly the same for all reinforced panels.

The amount of longitudinal cracking associated with the 7-inch uniform pavement section was considerably less than that on the 9-6-9 section for all panel lengths; but the difference was not as significantly large for the 15-foot panels as for the other panel lengths.

Table 11 shows the relationship of longitudinal cracking to pavement section for various panel lengths and expansion intervals of 120 and 125 feet. These data again indicate the advantages of the 7-inch uniform section over the 9-6-9 section in reducing longitudinal cracking. The cracking according to panel lengths based on these data is not similar to that previously shown above for the entire project since only 120 and 125-foot expansion intervals were included in Table 11.

The effects of restrained concrete on longitudinal cracking are indicated in Table 12 for various panel lengths with a 9-6-9 section. The relationship of longitudinal cracking to panel lengths was in the same order as shown in Table 10 for the entire project. The amount of cracking in the restrained concrete was also generally quite comparable, being slightly less for 15 foot panels, somewhat greater for 20 and 25 foot panels and slightly greater for the 30-foot panels. Averaging the feet of longitudinal cracks per mile for these panel lengths, the restrained concrete shows a value of 472 against 428 for the project. Thus it is indicated that restraint in the pavement did not significantly increase longitudinal cracking.

Faulted Joints

In July of 1944 there were five joints which showed faulting of $\frac{1}{4}$ inch or less. By 1950 there was a total of 195 joints which had faulted. Table 13 shows the distribution of this faulting on the basis of doweled and undoweled joints. It is apparent from these data that dowels were effective in reducing faulting on all sections except the 60-foot reinforced panels. There is no direct comparison available for the 60-foot panel length, but 40 percent of the joints were faulted even though dowels were present. It is also indicated that more faulting was associated with the 7-inch uniform pavement section than with the 9-6-9 section.

Table 14 shows the effect of panel length and expansion interval on faulting of 9-6-9 sections. Two things are of special interest in this table; first, the tendency of faulting to increase as panel lengths are increased; and second, the tendency of faulting to decrease as expansion intervals are increased.

Spalled Joints

The number of spalled joints has continued to increase with time. In 1944 only four spalled joints were noted, but this had increased to 50 in 1949 and to 81 in 1950. In addition there were a relatively large number of transverse cracks which were spalling. The progression of spalling has been more rapid on this project than on others since normal maintenance of joints was omitted or kept at a minimum due to the experimental nature of the project.

Table 15 shows the relation of spalled joints with expansion and contraction joint intervals. These data do not indicate any definite relationships; however, it appears that there was somewhat more spalling associated with the 7-inch uniform paving section than with the 9-6-9 section.

Corner Cracks

Only three corner cracks were noted in 1944 and all were breaks at the exterior edge of the 9-6-9 section One of the cracks was at an expansion joint and the other two at undoweled contraction joints. For the 9-6-9 section, a total of 17 internal and 12 external corner breaks developed by 1949. This increased to 28 internal and 26 external corner breaks by 1950. For the 7-inch uniform section, there was 1 internal and 2 external failures in 1949 and by 1950 this had increased to 2 internal and 6 external corner cracks. These data are shown in Table 16 in relation to expansion joint intervals. The over-all percentages for comparable sections of 9-6-9 and 7-inch uniform pavement indicate very little difference in the percent of total panels cracked; however, the distribution of the corner breaks was more nearly equal between internal and external corners for the 9-6-9 section. The 7-inch uniform section showed three times as many external corner breaks as internal.

Blow-Ups

The only blow-up on this project occurred in 1950, ten years after construction. This blow-up was in a 1,740-foot expansion interval and occurred at an untied, keyed, construction header joint located 510 feet from the expansion joint. This joint had shown evidence of eventual failure since 1944 when a slight raising of the joint was first noted.

A contraction joint, located 630 feet from the above blow-up, in a 1,245-foot expansion interval has shown partial failure due to compression since about three feet of the concrete on one side of this joint has been disrupted in the nature typical with blow-ups.

INDICATIONS AND CONCLUSIONS

The following general conclusions seem to be indicated by the data obtained during the first ten years of the life of this pavement.

1. Expansion joints are not necessary in rural pavements, except at fixed objects. They may be considered detrimental, if placed at close intervals because they permit excessive slab movement over a long period of time. The elimination of expansion joints will not cause excessive longitudinal stresses because of the compensatory effect of initial and subsequent shrinkage of the concrete.

2. Contraction joints should be placed at intervals of 15 feet in order to obtain the best over-all performance of the pavement slab from the standpoint of joint movement, cracking, warping, faulting and roughness.

3. The 7-inch uniform paving section appears to be superior to the 9-6-9 section but neither is entirely adequate. The 9-6-9 section was not thick enough through the center to cope with the subgrade and load stresses as indicated by longitudinal cracking. The 7-inch uniform section did not have adequate strength at the outer corners as compared to the inner corners as evidenced by the additional external corner breaks. It appears that a tapered 7-inch section, such as a 9-7-9, is indicated as being desirable.

4. Mesh reinforcement will not prevent cracking in slabs 30 feet or more in length.

5. Dowels are effective in reducing faulting at joints. Aggregate interlock may also be effective when expansion joints are eliminated and short panels, on the order of 15 feet in length, are used.

6. Metal seals, copper being used on this project, are not significantly effective in preventing vertical joint deformations.

7. The joint sealing material of the type manufactured by the Rubber Associates Company in 1940 proved to be more effective than the asphalt or latex-oil materials. However, this project has indicated that extended postponement of joint maintanance is detrimental to the pavement and even the better material did not adequately seal the joints after 10 years.

8. Expansion joint fillers, where used, should be non-extrusive in service and should prevent the leakage of water downward to the subgrade soil.

9. Concrete pavements tend to become gradually rougher with age. It is believed that this is due to the effects of loading, climate and subgrade rather than to pavement design features. Of these, the subgrade, the foundation of the pavement, offers the greatest opportunity for improving the stability, performance and service life of the entire road structure.

References

1. F.C. Lang, "Investigational Concrete Pavement in Minnesota", Proceedings, Highway Research Board, Vol. 20, p. 348 (1940).

2. F.C. Lang, "Invesgigational Concrete Pavement in Minnesota", Highway Research Board Research Report, No. 3B, p. 58 (1945).

3. J.A. Buchanan and A.L. Catudal, "Standardizable Equipment for Evaluating Road Surface Roughness", Proceedings, Highway Research Board, Vol. 20, p. 621 (1940).

Report on Experimental Project in Missouri

F. V. REAGEL, Engineer of Materials,
Missouri State Highway Commission

THE Missouri Investigational Project was constructed in the summer of 1941 on US 169 in the northwestern part of the state. It is on one of two main routes carrying traffic from Kansas City and St. Joseph, Missouri to Des Moines, Iowa. A description of this project was published in the 1941 Proceedings of the Highway Research Board and a report of subsequent measurements and observations was included in the Highway Research Board's Research Report No. 3B, issued in 1945. The pavement is now 10 years old and the information collected and analyzed since 1945 is presented in this 10 year progress report.

TRAFFIC

The traffic carried by this project during its life has been rather light in comparison with that using most main highways. Until wartime restrictions were removed in 1945, traffic was very light. Thereafter there was a sharp increase as shown in the following tabulation:

Year	Total Cars and Trucks
1941	761
1942	605
1943	544
1946	943
1947	1052
1948	1052
1950	1330

No traffic classification has ever been made within the limits of this investigational project, however, a classification that could be considered representative was made on this same route in 1950 at a location about six miles south of the project. The distribution by vehicle types at this point was found to be as follows:

Passenger cars, 68. 6 percent; panel and pickup trucks, 8. 9 percent; single unit trucks (6 tires) 19. 1 percent; trailer combinations, 2. 5 percent; busses, 0. 9 percent.

JOINT MOVEMENTS

As was stated in the previous progress report, inconsistencies in joint movements, probably caused by variations in curling of the slabs, subgrade friction, moisture content, infiltration of inert materials into the joint spaces and other factors which could not be measured and evaluated, existed even on supposedly similar sections. The data in this report are treated as in the first progress report, without correction for these unmeasured variations. Measurements at joints, having adjacent cracked slabs, were eliminated from calculations for average joint movements.

Cross-joint measurements were made to determine daily and seasonable changes on August 15 and November 15, 1945, and on February 9 and May 16, 1946. Residual movement measurements were made during August in 1945, 1946, 1947, and 1951.

Daily Joint Measurements

The average daily joint movements are shown in Table 1. The measurements were taken at the minimum and maximum slab temperatures.

Contraction Joints. In the series of joint measurements made from August 15, 1945, through May 16, 1956, it was found that the average daily movement at contraction joints spaced at 25 feet was approximately the same regardless of the spacing of the expansion joints. In sections with expansion joints spaced at 125 feet, and intermediate contraction joints spaced at 25 feet, the daily movement at the contraction joints with dowels was approximately 40 percent less than the movement at the joints without dowels. This is probably an indication that the dowels are not functioning properly and are restraining free movement of the slabs. Restraint could be caused by rust bonding or poor alignment of the dowels.

TABLE 1

DAILY CHANGES IN JOINT OPENINGS [a]

P.R.A. Section Number	Slab Length (Ft.)	Expan. Jt. Spacing (Ft.)	Load Transfer Con. Jts.	Load Transfer Expan. Jts.	Rein-force-ment	Pav. X-Sec. (In.)	Av. Change in[b] Joint Opening Contr.	Av. Change in[b] Joint Opening Expan.	Av. Temp. Change °F.
1	25	No expan. Jts.	None		None	9-7-9	24		18
2 & 2R	25	800	None	Translode	None	9-7-9	21	15	16
3 & 3R	25	400	None	Translode	None	9-7-9	18	17	17
5 & 5R	25	125	Dowels	Translode	None	9-7-9	15	52	17
6 & 6R	60	120	Dowels	Translode	70-lb. mesh	9-7-9	20	69	18
6M & 6MR	60	120	Dowels	Translode	43-lb. mesh	9-7-9	13	71	18
7 & 7R	25	125	None	None	None	7-unif.	24	28	17
7M & 7MR	25	125	Dowels	Translode	None	10-8-10 [c]	13	60	18
8M & 8MR	60	No expan. Jts.	Dowels		43-lb. mesh	9-7-	33		19
10M	25	125	None	None	None	8-unif.	23	24	18
11M	25	125	None	None	None	9-unif.	24	24	18

[a] In thousandths of an inch.
[b] Average of measurements made on 8-15 and 11-15, 1945, and 2-9 and 5-16, 1946.
[c] Actually 9.8-7.8-9.8

The average daily movement at contraction joints was essentially the same for 25 and 60 foot slabs where load transfer devices were used in all joints and expansion joints were spaced at 120 and 125 feet. This is contrary to expectations as it would seem that the movement at contraction joints spaced at 25 feet should have been less than at those spaced at 60 feet where expansion joints are approximately the same distance apart.

In those sections with 60 foot slab lengths, the daily movement at the doweled contraction joints in sections having no expansion joints was twice as great as at the doweled contraction joints in sections containing alternate expansion joints. Thus it appears, in this case, that the dowels are not restraining slab movement but that possibly restraint is offered by the "Translode" device at the expansion joints.

The average daily movement at contraction joints in the three sections having uniform pavement thicknesses of 7, 8, and 9 inches was practically the same. It should be particularly noted that in 25 foot slabs with expansion joints every 125 feet without load transfer devices in either the contraction or expansion joints, the daily movement of the contraction joints is nearly identical with that of the expansion joints. At this age the slabs have shifted into positions of equilibrium where movement at both expansion and contraction joints is unrestrained.

Expansion Joints. The average daily movements at expansion joints varied with lengths of the slabs, the spacing of the expansion joints and whether or not dowels were used in the intermediate contraction joints. For 25 foot slabs having undowelled contraction joints, the daily movement at expansion joints spaced at 125 feet was about 50 percent greater than at the expansion joints which were spaced at 400 and 800 feet. There was practically no difference between the average daily movement of the expansion joints which were spaced at 400 and at 800 feet.

In sections with expansion joints at 125 feet and contraction joints at 25 feet, the average daily movement at the expansion joints with load transfer and with intermediate doweled contraction joints was over twice the movement at the expansion joints where no load transfer devices were used in either expansion or contraction joints. This is another indication that the dowels are probably restraining movement of the slabs since any restraining force at the contraction joints should cause a greater movement at the expansion joints.

The daily movement at the expansion joints which were spaced at 120 feet with one intermediate doweled contraction joint was approximately 25 percent greater than at those spaced at 125 feet with intermediate doweled contraction joints at 25 foot spacing.

There was practically no difference in the daily movement at expansion joints in the

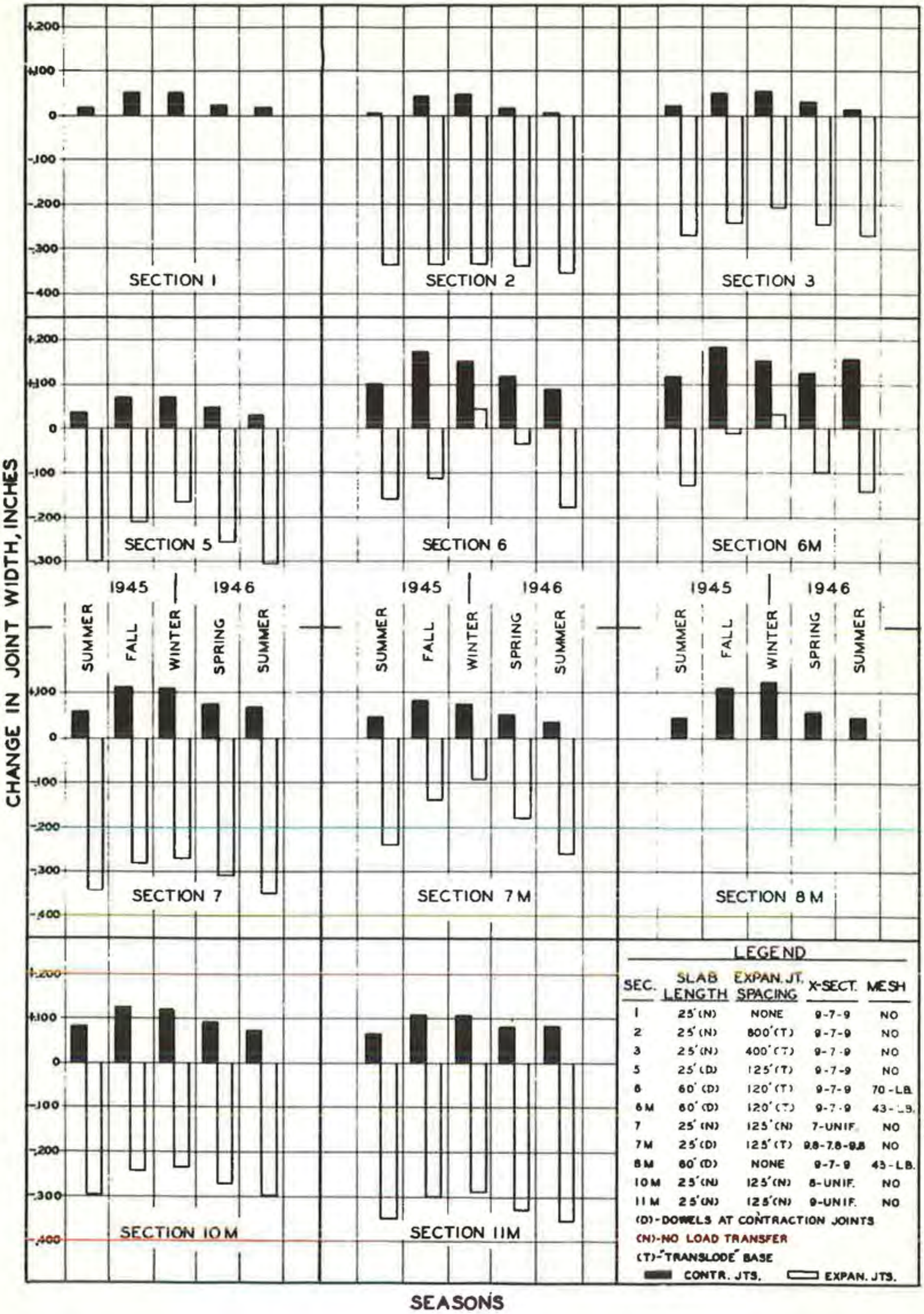


Figure 1. Seasonal changes in joint openings.

TABLE 2

RESIDUAL CHANGES IN JOINT OPENINGS[a]

P. R. A. Section Number	Slab Length (Ft.)	Expan. Jt. Spacing (Ft.)	Load Transfer		Rein-force-ment	Pav. X-Sec. (In.)	Contraction Joints							Expansion Joints						
			Contr. Jts.	Expan. Jts.			Aug. '42	Aug. '43	Aug. '44	Aug. '45	Aug. '46	Aug. '47	Aug. '51	Aug. '42	Aug. '43	Aug. '44	Aug. '45	Aug. '46	Aug. '47	Aug. '51
1	25	No. Exp. jts.	None		None	9-7-9	+3	+12	0	+21	+18	+16	+6							
2 & 2R	25	800	None	Translode	None	9-7-9	+13	+14	+2	+10	+5	+6	-3	-307	-333	-337	-326	-324	-320	-303
3 & 3R	25	400	None	Translode	None	9-7-9	+23	+19	+13	+15	+12	+12	+6	-202[b]	No Read.	-276	-354	-355	-354	-351[c]
5 & 5R	25	125	Dowels	Translode	None	9-7-9	+14	+23	+25	+39	+31	+34	+38	-203	-229	-275	-248	-261	-264	-259
6 & 6R	60	120	Dowels	Translode	70-lb. mesh	9-7-9	+68	+63	+72	+105	+91	+95	+89	-101	-157	-172	-178	-208	-225	-241
6M & 6MR	60	120	Dowels	Translode	43-lb. mesh	9-7-9	+66	+85	+148	+187	+174	+180	+188	35	-121	-103	-137	-187	-207	-260
7 & 7R	25	125	None	None	None	7-Unif.	+39	+56	+54	+86	+71	+73	+78	-206	-293	-316	-291	-310	-315	-318
7M & 7MR	25	125	Dowels	Translode	None	10-8-10[d]	+40	+41	+42	+49	+45	+45	+48	-124	-201	-255	-282	-308	-310	-320
8M & 8MR	60	No. Exp. jts.	Dowels		43-lb. mesh	9-7-9	+25	+29	+29	+38	+38	+42	+45							
10M	25	125	None	None	None	8-Unif.	+59	+82	+80	+93	+84	+93	+79	-217	-284	-299	-300	-316	-317	-318
11M	25	125	None	None	None	9-Unif.	+49	+66	+61	+69	+62	+66	+64	-229	-322	-354	-342	-346	-340	-355

[a] Each tabular figure, in thousandths of an inch, is the average cumulative change in joint openings from the base measurement made in August 1941. Plus values indicate joint openings and minus values joint closures.
[b] No reading in August, 1941. Used reading of July 22, 1941, as base measurement for all expansion joints on sections 3 & 3R.
[c] Measurement is for Sec. 3R only
[d] Actually 9.2-7.4-9.8

slabs reinforced with 43 pound and 70 pound mesh.

Seasonal Joint Measurements

Seasonal movements of contraction and expansion joints, as determined during the 1945-46 seasonal cycle, are shown in Figure 1. These are the only seasonal measurements that have been obtained since those which were presented in the first progress report.[1]

Contraction Joints. During this cycle the magnitude of the seasonal movement at contraction joints without dowels was approximately the same for all 25 foot slabs, regardless of the spacing of the expansion joints. The movement at the doweled contraction joints bounded by 60 foot slabs was nearly twice that occurring at joints between 25 foot slabs. The maximum movement of contraction joints from datum was greatest for those bounded by 60 foot slabs and having alternate expansion joints.

Based upon the conclusions of Sutherland and Cashell[3], and upon measurements from datum, maximum openings of contraction joints spaced at 25 feet, even with remotely spaced expansion joints, were of such magnitude that aggregate interlock could not be considered to be effective for stress control or load transfer during cold seasons, since average openings exceeded 0.04 inches.

Expansion Joints. In sections with contraction joints at 25 feet and expansion joints at 125 feet, the seasonal movement of the expansion joints during this cycle was about twice as great where dowels were used in the contraction joints. Thus it again appears that dowels at the contraction joints are not permitting free movement of the slabs.

In Section 2, with undowelled contraction joints spaced at 25 feet and containing expansion joints with load transfer spaced at 800 feet, there was practically no change in the expansion joint openings throughout the seasonal cycle. The movement was practically all confined to the contraction joints.

The movement at expansion joints spaced at 120 feet with one intermediate doweled contraction joint, was approximately 60 percent greater than that at expansion joints spaced at 125 feet with 4 intermediate doweled contraction joints at 25 feet.

Variations in cross-section of slabs and in weight of mesh had little or no effect on the seasonal changes in joint openings.

Residual Changes in Joint Openings

Average residual joint movements are shown in Table 2. Values for all years in which measurements were made are included to facilitate observation of trends.

Contraction Joints. After the pavement was four years old, there was very little change in the residual openings of the contraction joints. This indicates that slabs had shifted into positions where length changes during a yearly cycle of exposure caused practically no change in the residual openings.

At the age of ten years the average residual opening of contraction joints, without dowels and spaced at 25 feet, was appreciably greater for 125 foot spacing of expansion joints than for the longer spacings, the average being about 25 times greater.

[1] Highway Research Board, Research Report No. 3B

In sections with slabs of 25 foot length and with expansion joints spaced at 125 feet, the average residual opening at contraction joints without dowels was approximately 80 percent greater than at contraction joints with dowels. This is another indication that improper dowel action may be preventing the free movement of slabs.

Residual openings of contraction joints were more erratic and were greater for the longer contraction joint spacing, but the longer slabs were all reinforced while the shorter slabs were not. In 60 foot slabs with alternate expansion and contraction joints, the average residual opening of contraction joints, for some inexplicable reason, was greater with 43-lb. mesh than with 70-lb. mesh.

Expansion Joints. During the early age of the concrete pavement, progressive closure of expansion joints was noted for all spacings. Closure apparently ceased on most sections after 5 years; however, the closure of expansion joints in sections with alternate expansion and contraction joints at 60 foot spacing still seems to be continuing at the age of ten years.

In sections with slabs of 25 foot length, and with the spacing of the expansion joints varying from 125 to 800 feet, the residual movement toward closure was of approximately the same magnitude; that is, about one-third of an inch. There is, however one exception. In sections with slabs of 60 foot length, with alternate expansion and contraction joints, the expansion joints have closed approximately one quarter of an inch in 10 years.

A comparison of total contraction joint movements with total expansion joint movements is of interest. In sections with slabs of 25 foot length and with expansion joints spaced at 125 feet, the total opening of the intermediate contraction joints in which dowels were used does not approach the total closure of the adjacent expansion joint, while where dowels were not used, the total opening of the intermediate contraction joints closely approaches the closure of the adjacent expansion joint. This again indicates that the load transfer devices are preventing the free movement of the slabs.

FAULTING MEASUREMENTS

In the summer of 1951 differential measurements were made at four places at each joint, one about 6 inches on each side of the center line and one about 16 inches from each edge of the pavement. These places were chosen in order that no interference

TABLE 3

DISTRIBUTION OF JOINT FAULTING BY GROUPS AS OF AUGUST 1951

(Percent)

P. R. A. Section			Left Lane						Right Lane							
	Joints		Left Point			Right Point			Joints		Left Point			Right Point		
Number	Number	Type[1]	1/8" or less	1/8"-1/4"	+1/4"	1/8" or less	1/8"-1/4"	+1/4"	Number	Type[1]	1/8" or less	1/8"-1/4"	+1/4"	1/8" or less	1/8"-1/4"	+1/4"
1	248	CN	91	8	1	95	5	-	250	CN	94	6	-	93	6	1
2 & 2R	184	CN	85	14	1	94	6	-	182	CN	96	4	-	90	9	1
	6	ET	83	17	-	100	-	-	6	ET	84	16	-	100	-	-
3 & 3R	203	CN	86	13	1	95	5	-	206	CN	94	6	-	91	9	-
	14	ET	100	-	-	100	-	-	14	ET	100	-	-	100	-	-
5 & 5R	87	CD	99	1	-	100	-	-	89	CD	100	-	-	100	-	-
	22	ET	86	14	-	95	5	-	22	ET	100	-	-	100	-	-
6 & 6R	21	CD	100	-	-	100	-	-	20	CD	100	-	-	100	-	-
	19	ET	89	11	-	100	-	-	17	ET	100	-	-	87	13	-
6M & 6MR	23	CD	100	-	-	100	-	-	23	CD	100	-	-	96	4	-
	24	ET	96	4	-	96	4	-	24	ET	92	8	-	100	-	-
7 & 7R	92	CN	78	17	5	86	12	2	94	CN	93	6	1	87	11	2
	24	EN	54	38	8	92	-	8	24	EN	92	4	4	65	25	10
7M & 7MR	87	CD	100	-	-	100	-	-	92	CD	100	-	-	100	-	-
	22	ET	100	-	-	100	-	-	22	ET	95	5	-	100	-	-
8M & 8MR	41	CD	98	2	-	98	2	-	42	CD	98	-	2	98	2	-
10M	38	CN	92	8	-	92	8	-	38	CN	97	3	-	92	8	-
	10	EN	70	10	20	80	20	-	10	EN	40	50	10	40	40	20
11M	36	CN	83	12	5	95	5	-	36	CN	97	3	-	95	5	-
	9	EN	78	22	-	55	34	11	9	EN	22	78	-	22	45	33

[1] Joint Types

CN-Dummy contraction without mechanical means of load transfer.

CD-Dummy contraction with 7/8" x 16" bars spaced 12", 1/2 length coated with red lead and motor graphite.

EN-Expansion without mechanical means of load transfer and with premoulded filler.

ET-Translode type with 2" x 3" x 1/4" angles and premoulded filler.

TABLE 4
SUMMARY OF DIRECTIONAL FAULTING
AS OF AUGUST 1951
(Percent)

P.R.A. Section Number	Left Lane								Right Lane							
	Joints		Left Point			Right Point			Joints		Left Point			Right Point		
	Number	Type[a]	-	0	+	-	0	+	Number	Type[a]	-	0	+	-	0	+
1	248	CN	61	26	13	57	29	14	250	CN	46	36	18	54	32	14
2 & 2R	184	CN	50	34	16	47	34	19	182	CN	51	31	18	50	30	20
	6	ET	72	14	14	58	28	14	6	ET	57	14	29	14	43	43
3 & 3R	206	CN	56	28	16	49	34	17	206	CN	49	32	19	49	31	20
	14	ET	71	22	7	43	36	21	14	ET	64	29	7	71	22	7
5 & 5R	87	CD	28	44	28	46	47	7	89	CD	20	60	20	44	45	11
	22	ET	73	18	9	45	55	--	22	ET	50	36	14	59	32	9
6 & 6R	21	CD	19	76	5	35	55	10	20	CD	25	65	10	50	44	6
	19	ET	74	10	16	38	56	6	17	ET	39	44	17	56	31	13
6M & 6MR	23	CD	35	35	30	57	39	4	23	CD	30	57	13	61	26	13
	24	ET	67	33	--	46	46	8	24	ET	42	42	16	38	54	8
7 & 7R	92	CN	58	20	22	62	26	12	94	CN	46	27	27	57	27	16
	24	EN	62	8	30	50	30	20	24	EN	62	25	13	66	17	17
7M & 7MR	87	CD	32	58	10	60	39	1	92	CD	24	64	12	26	55	19
	22	ET	41	45	14	50	32	18	22	ET	45	37	18	54	32	14
8M & 8MR	41	CD	44	46	10	44	37	19	42	CD	47	48	5	60	31	9
10M	38	CN	53	26	21	71	21	8	38	CN	45	34	21	53	21	26
	10	EN	40	--	60	30	30	40	10	EN	90	--	10	90	--	10
11M	36	CN	53	22	25	61	28	11	36	CN	53	33	14	47	23	30
	9	EN	33	11	56	22	11	67	9	EN	67	11	22	78	--	22

[a] Joint Types

CN - Dummy contraction without mechanical means of load transfer.
CD - Dummy contraction with ⅞" x 16" bars spaced 12 ", ½ length coated with red lead and motor graphite.
EN - Expansion without mechanical means of load transfer and with premoulded filler.
ET - Translode type with 2"x3"x¼" angles and premoulded filler.

Faulting

\- Normal Faulting
\+ Reverse Faulting

in measurements would be obtained by slight spalling and scaling of the surface along the center joint or by any irregularities of the surface due to lip curb construction.

Measurements were obtained by using a device that consisted of a sliding rod in a pipe, offset on a base angle to permit differential readings across a joint. The indicator of the rod was adjusted to read zero on the scribed scale on the pipe at eye level when the device was placed on a plane surface. Readings were recorded to the nearest one-sixteenth of an inch. Places selected for measurement were cleaned of all dirt and extruded joint filler to permit fairly accurate determinations.

Normal faulting as used in this discussion, denotes that the surface of the forward slab, (the slab ahead when facing in the direction of traffic) is lower than the surface of the rear slab. It is the condition normally found when subgrade pumping takes place. Reverse faulting denotes the opposite condition.

The results of faulting measurements are shown in Tables 3, 4, and 5. Table 3 shows the percent of faulting in three groups irrespective of the type of faulting. Table 4 shows the percent of directional faulting. Table 5 shows the average amount of faulting per joint, irrespective of the type of faulting. Faulting measurements at joints bounded by cracked slabs were eliminated from the averages.

In Table 3 it may be seen that the dowels have been very effective in preventing faulting at contraction joints. There are also indications that dowelling of contraction

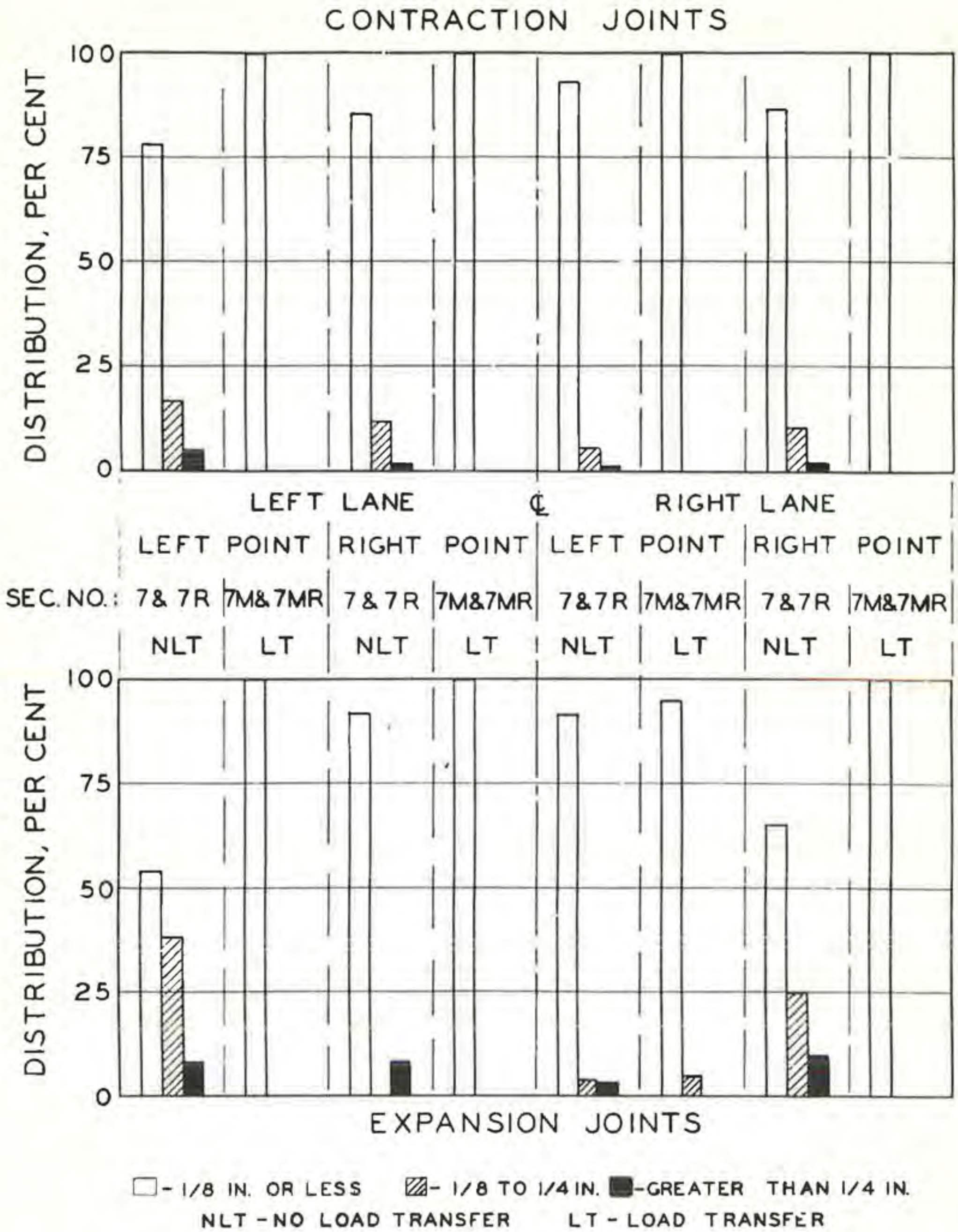


Figure 2. Comparison of faulting at joints without and with mechanical means of load transfer.

joints spaced at 25 feet, is more essential when expansion joints are spaced at 125 feet than when expansion joints are 800 feet or more apart. Translode bases at expansion joints are seen to be very effective in the prevention of faulting. The greatest amount of normal type faulting observed at any joint was $^{13}/_{16}$ of an inch, and occurred at a contraction joint.

By means of Figure 2 faulting at joints containing load-transfer devices may be compared with that at joints which do not have this feature. The figure indicates that

TABLE 5

SUMMARY OF MAGNITUDE OF JOINT FAULTING

(Average Unit Per Joint)[1]

P. R. A. Section Number	Joint Spacing	Expan. Joint Spacing	Dowels at Contr. Joint	X-Sec.	Left Lane						Right Lane					
					Expansion Joints			Contraction Joints			Expansion Joints			Contraction Joints		
					No.	Lt. Pt.	Rt. Pt.	No.	Lt. Pt.	Rt. Pt.	No.	Lt. Pt.	Rt. Pt.	No.	Lt. Pt.	Rt. Pt.
	feet	feet		inches												
1	25	None	No	9-7-9	-	-	-	248	1.15	1.02	-	-	-	250	0.91	1.06
2 & 2R	25	800	No	9-7-9	6	1.17	0.67	184	1.17	0.93	6	1.67	0.67	182	1.03	1.09
3 & 3R	25	400	No	9-7-9	14	1.22	0.64	206	1.24	0.90	14	0.72	0.86	206	1.06	1.15
5 & 5R	25	125	Yes	9-7-9	22	1.46	0.68	87	0.61	0.55	22	0.90	0.90	89	0.40	0.58
6 & 6R	60	120 (a)	Yes	9-7-9	19	1.37	0.53	21	0.34	0.45	17	0.56	1.05	20	0.35	0.55
6M & 6MR	60	120 (b)	Yes	9-7-9	24	0.96	0.91	23	0.87	0.62	24	0.96	0.55	23	0.48	0.97
7 & 7R	25	125 (c)	No	7-Unif.	24	2.54	1.38	92	1.72	1.29	24	1.21	2.42	94	1.06	1.33
7M & 7MR	25	125	Yes	9.8-7.8-9.8	22	0.69	0.91	87	0.51	0.72	22	0.87	0.96	92	0.37	0.39
8M & 8MR	60	None(b)	Yes	9-7-9	-	-	-	41	0.68	0.81	-	-	-	42	0.67	0.74
10M	25	125 (c)	No	8-Unif.	10	2.30	1.30	38	1.16	1.24	10	2.70	3.40	38	0.76	1.11
11M	25	125 (c)	No	9-Unif.	9	1.67	2.33	36	1.61	1.00	9	2.89	4.11	36	0.89	1.14

[1] Unit is $\frac{1}{16}$ of an inch

(a) 70-lb. mesh reinforcement.
(b) 43-lb. mesh reinforcement.
(c) Without means of load transfer.

the load transfer devices have thus far been effective in the prevention of faulting. Although a difference is indicated in the degree of faulting, the difference in the rideability of these two sections is hardly distinguishable. Several reasons account for this fact. One is that the magnitude of faulting was small and another was that the percentage of normal faulting was far greater than that of reverse faulting. Under these conditions very little impact is noticeable at normal speeds. The greatest magnitude of reverse faulting noted was ¼ inch at an expansion joint which had no built-in load transfer.

TABLE 6

CONDITION SURVEY AS OF AUGUST, 1951

P. R. A. Section	No. of Slabs	No. Slabs Cracked	Cracks per Slab	Unbroken Slab Length (Ft.)	Map Cracking Sq. Ft.	Remarks
1	250	2	.004	24.9	1	
2 2R	192	7	.029	24.3	2	Moderate checking in 6 slabs
3 3R	224	4	.014	24.7	4	
5 5R	120	13	.085	23.0	0	Moderate checking in 16 slabs
6 6R	50	8	.100	54.5	109	
6M 6MR	50	3	.080	55.6	5	
7 7R	120	2	.012	24.7	0	One interior corner break
7M 7MR	120	8	.050	23.8	0	
8M 8MR	50	9	.190	50.4	0	Two interior corner breaks, one spalled joint
10M	45	0	.000	25.0	50	
11M	46	0	.000	25.0	0	

Table 4 shows that percentages of normal faulting were greater than the percentages of reverse faulting at contraction joints either with, or without, dowels. There was some reverse faulting at expansion joints but the majority of the faulting was of the normal type. The effects of load transfer devices may also be noted; the percentage of faulted joints, whether contraction or expansion joints, being small where such devices were used.

In Table 5 it may be seen that the magnitude of average faulting per joint was greater for those expansion joints which have no means of mechanical load transfer. This was also true for contraction joints without dowels. Generally the magnitude of average faulting per joint was slightly greater near the outside edge of the pavement than near the centerline. This does not necessarily indicate that tilting of all individual slabs has taken place because the tabular values are averages. However, some slight tilting of slabs was observed, usually in areas where fill settlements or frost heaving had occurred.

While faulting at joints cannot presently be considered a major imperfection of this project, the consistent difference between protected and unprotected joints of all types induces a strong suspicion that the magnitude of the difference would have been much more pronounced under a larger volume of heavier trucks.

CONDITION SURVEY

The results of the latest condition survey are shown in Table 6. The average number of cracks per slab was greater for slabs of 60 foot length than for slabs of 25 foot length; however, the unbroken slab length was greater for 60 foot slabs. In 60 foot slabs, the number of cracks per slab was greater for sections without expansion joints than for sections with alternate contraction and expansion joints. It should be noted that dowels were used in all contraction joints between 60 foot slabs.

In slabs of 25 foot length with expansion joints spaced at 125 feet, the cracks per slab were greater where dowels were used at contraction joints. With this same spacing, but without dowels at contraction joints, there was no evidence of cracking in the thicker slabs. Only one faulted crack was observed in all of the test sections. It was in Section 7MR in a medium size cut where a frost boil had occurred during the first winter.

A slight amount of moderate checking appeared during the early life of the pavement. Some structural cracking has emanated from larger checks but the amount is very small. No map cracking has developed in the checked areas to date.

Map cracking was very slight, both in extent and intensity and was confined to areas with a high water table or poor drainage. There was no longitudinal cracking, except for a few small checks. Three corner breaks have occurred, one in a 25 foot slab without reinforcement and a 7-inch uniform cross section, and two in 60 foot slabs with 43-lb. reinforcement and a 9-7-9 inch cross section. Exterior corner breaks have not occurred to date.

Generally the joints were in good condition. There was little to no spalling at transverse joints or cracks, however, there was extensive slight spalling and scaling along the unedged longitudinal center joint which occurred during the first few years and has not subsequently progressed. Spalling had occurred at only one joint and that was at a dowelled contraction joint in Section 8. The nature of the spall indicated that it was caused by entrance of incompressible material at the surface of the joint.

Extrusion of the premoulded sponge rubber joint filler, AASHO M58, Type III, was noted at the expansion joints on Sections 5, 5R, 6, 6R, 6M and 6MR, while this was not evident on other sections where the same type of joint was used. The filler has extruded from ½ to 1 inch above the surface of the pavement and was torn by traffic. The extent of extrusion is not in agreement with the magnitude of residual joint closure since the smaller closures showed the greater amount of extrusion. Variations in the amounts of deleterious material entering these joints, and greater daily movement of the joints, could have caused this inconsistency. The joints have been resealed on several occasions, but apparently, to be effective, sealing should be repeated every winter.

The last six expansion joints of the project were poured full depth with an asphalt-latex joint filler which met the requirements of a specification furnished by the Oklahoma Highway Department. The asphalt-latex mixture was still pliable after ten years of weathering and was adhering to some extent to the walls of the joints. Some slight weathering was evident in the top ⅜ inch. There was little evidence that the asphalt-latex joints have been resealed since they were originally installed. Practically all the contraction joints sealed with Plastex T. A. filler have had to be resealed on several occasions during the past ten years.

CONCLUSION

Developments on this project to date do not warrant drawing conclusions as to the various design features studied.

Investigational Concrete Pavement in Oregon

G. S. PAXSON, Bridge Engineer
Oregon State Highway Department

THE section of pavement covered by this report is one of the six sections in six states built under cooperative agreements with the Bureau of Public Roads in 1940 and 1941. Two progress reports have previously been made.[1, 2] In these previous reports the location and details of construction were described in detail. For convenience the data are repeated here.

The Lombard Street-Killingsworth Street Section of the Northeast Portland Secondary Highway is located on a high river bench sloping slightly and evenly toward the north. The highway follows along the bench at practically a level grade with generally a light cut on the south and a low fill on the north. At only two places do the fills or cuts on center line exceed 4 feet in depth. At these two points small drainage courses are crossed with a maximum fill depth of 22 feet. The embankment was placed in 6-inch layers and compacted with hauling equipment. The grading was done in 1939 and had practically two years to settle before the pavement was placed in 1941.

The soil over the entire project is classified as A-4. It had a liquid limit of from 23 to 27 and a plasticity index of zero except for two sections where the plasticity index was 3. The soil analysis given in Table 1 is representative of the soil over the entire project.

The details of the project followed the outline specified by the Bureau of Public Roads[3] for the investigation except that their Section No. 2 was omitted. The dimensions of the sections and the type of pavement and joint treatment are given in Table 2.

The Lombard Street-Killingsworth Street Section had other advantages that influenced the selection. It is partly within the City of Portland and serves as a by-pass route for US 30 traffic from the east. It carries a relatively heavy volume of travel and the percentage of heavy trucks is greater than on most sections of highway. The average daily traffic for the ten-year period from 1941 to 1950, inclusive, is shown in Table 3.

The project was set up primarily to observe the effect of expansion joint spacing on the movements of expansion and contraction joints. The joints were built with and without load transfer devices with the expectation that an indication of the value of such devices might be observed.

Present Condition of the Pavement

The pavement after ten years of service is in excellent condition. This is true of all six of the test sections. The general excellence of the pavement makes it difficult, if not impossible, to compare the relative merit of the different joint and load-transfer arrangements. Except for the two crossings of minor drainage, no unequal settlement has occurred. At these two points a small amount of settlement has taken place, but without breakage of the pavement slabs. There are nine cracks in the entire length. In all cases the cracks are across one lane only and stop at the longitudinal joint. No specific cause can be assigned to any of these cracks. There seems to be no relationship between the position of the cracks and the distance between expansion joints or to the load transfer at the joints.

Photographs of one of these cracks were taken in 1943, when it was first observed, in 1945, and again in 1950. Figure No. 1 shows the development in eight years.

---

[1] G.S. Paxson, "Investigational Concrete Pavement in Oregon," Proceedings, Highway Research Board, Vol. 21, P. 147.

[2] G.S. Paxson, "Investigational Concrete Pavement in Oregon," Highway Research Board Report No. 3B, 1945.

[3] E. F. Kelley, "History and Scope of Cooperative Studies of Joint Spacing in Concrete Pavements," Proceedings, Highway Research Board, Vol. 20, P. 333.

TABLE 1

SOIL ANALYSIS

	%		%
F.M.E.	20	Larger than 2.0mm.	0.0
C.M.E.	9	Coarse sand, 2.0 to 0.25 mm.	9.8
L.L.	24	Fine sand, 0.25 to 0.05 mm.	36.7
P.L.	0	Silt, 0.05 to 0.005 mm.	37.4
P.I.	0	Clay, smaller than 0.005 mm.	6.1
S.L.	0	Colloids, smaller than 0.001 mm.	5.3
L.S.	0		
S.G.	2.63	Group A-4	

The spalling at the contraction joint corners has not significantly increased in the six years since the previous report was made. This spalling is certainly caused by improper placing of the elastic material in the upper part of the contraction joint. This spalling is only on the surface and does not affect the lower two thirds of the pavement slab.

Width Change of Joints

Section No. 1 is a mile in length without expansion joints, so that expansion of the concrete can only be accommodated by slab movement at the two ends of the section. Section No. 3 is 2,430 feet in length but is divided by expansion joints into six subsections, each 405 feet in length. Sections No. 4, 5, 6 and 7 are 1,200 feet in length, each divided into 10 subsections 120 feet in length. In all sections except No. 6, the reinforced section, contraction joints are at 15-foot intervals. In Section No. 6 each 120-foot subsection has a contraction joint at its mid-length dividing it into two 60-foot panels.

Measuring stations were installed at each end, at the mid-point, and at the quarter points of Section No. 1; at each end and at the mid-point of two of the subsections of Section No. 3; at each end of five of the subsections in Sections No. 4, 5, 6 and 7. Gauge points were placed to measure the change in width of all expansion joints in the selected subsections listed above. Gauge points were also placed at selected contraction joints in Sections No. 1 and No. 3 and at all contraction joints in two subsections of Sections No. 4, 5, and 7 and at the single contraction joint in each of the five subsections of Section No. 6.

TABLE 2

ARRANGEMENT OF EXPERIMENTAL SECTIONS

Section No.	Length	Thickness	Metal Reinforcement	Expansion Joints Spacing	Expansion Joints Load Transfer	Contraction Joints Spacing	Contraction Joints Load Transfer
	(feet)	(inches)		(feet)		(feet)	
1	5,280	9-7-9	None	At ends	Dowels	15	None
3W	2,430	9-7-9	None	405	Dowels	15	None
3E	2,430	0-7-9	None	405	Dowels	15	None
4W	1,200	9-7-9	None	120	Dowels	15	None
4E	1,200	9-7-9	None	120	Dowels	15	None
5W	1,200	9-7-9	None	120	Dowels	15	Dowels
5E	1,200	9-7-9	None	120	Dowels	15	Dowels
6W	1,200	9-7-9	Mesh	120	Dowels	60	Dowels
6E	1,200	9-7-9	Mesh	120	Dowels	60	Dowels
7W	1,200	8 uniform	None	120	None	15	None
7E	1,260	8 uniform	None	120	None	15	None

The measuring stations at the section ends are not affected by pavement movement and provide means by which the total elongation of the subsections can be determined. The gross elongation of the subsections is also represented by the closure of the ex-

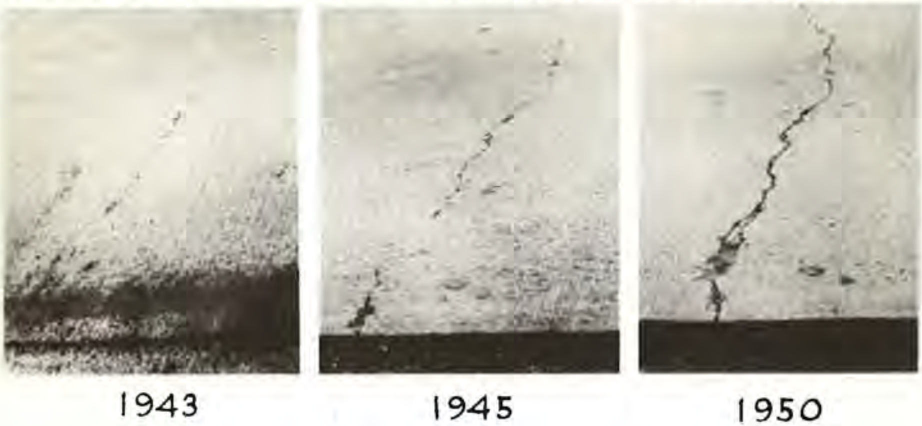


Figure 1

pansion joints. In Figure No. 2 is shown the gross elongation or expansion joint closure of the six sections. An interesting feature of these graphs is the similarity of the five unreinforced sections both as to pattern and amount of closure. The expansion joints, all of which were originally 0.75 inch in width, have closed up until now after ten years of service they are practically half their original width. The closure of the joints at each end of the mile-long section is no greater than the closure at the ends of the 120-foot sections.

The rate of closure for all unreinforced sections was rapid in the first year and a half and then decreased. The closing of the expansion joints has continued during the entire ten-year period, however, and there is no indication that equilibrium will be established short of complete closure.

The reinforced section, No. 6, presents a slightly different pattern. After the first year and a half until 1949, the joint movement repeated itself with but little variation other than that due to the temperature at the particular day on which the measurement was taken. During the last two years additional joint closure has occurred. It is probable that the joints will eventually close as is indicated for the joints in the unreinforced sections.

TABLE 3

TRAFFIC DATA-LOMBARD-KILLINGSWORTH SECTION

			Daily Truck Traffic			
	Total Daily Traffic		Average		Maximum	
Year	Average	Maximum	Light	Heavy	Light	Heavy
1941	3810	6400	169	220	231	316
1942	4170	6210	184	262	263	390
1943	4200	5660	205	277	277	385
1944	3865	4830	212	290	256	357
1945	4440	5550	257	341	299	418
1946	5210	7720	276	411	358	521
1947	5770	8760	294	368	370	490
1948	6345	10780	324	339	510	510
1949	7150	12155	361	379	571	574
1950	7300	11750	406	439	510	580
10-yr.-averages of all	5226	7981	269	332	364	454

The joint closure at present is about half that of the unreinforced sections of the same length, but the seasonal variations are greater. This is to be expected because of the contraction joint arrangement. There is only one contraction joint in each subsection in the reinforced section, while there are seven such joints in the unreinforced

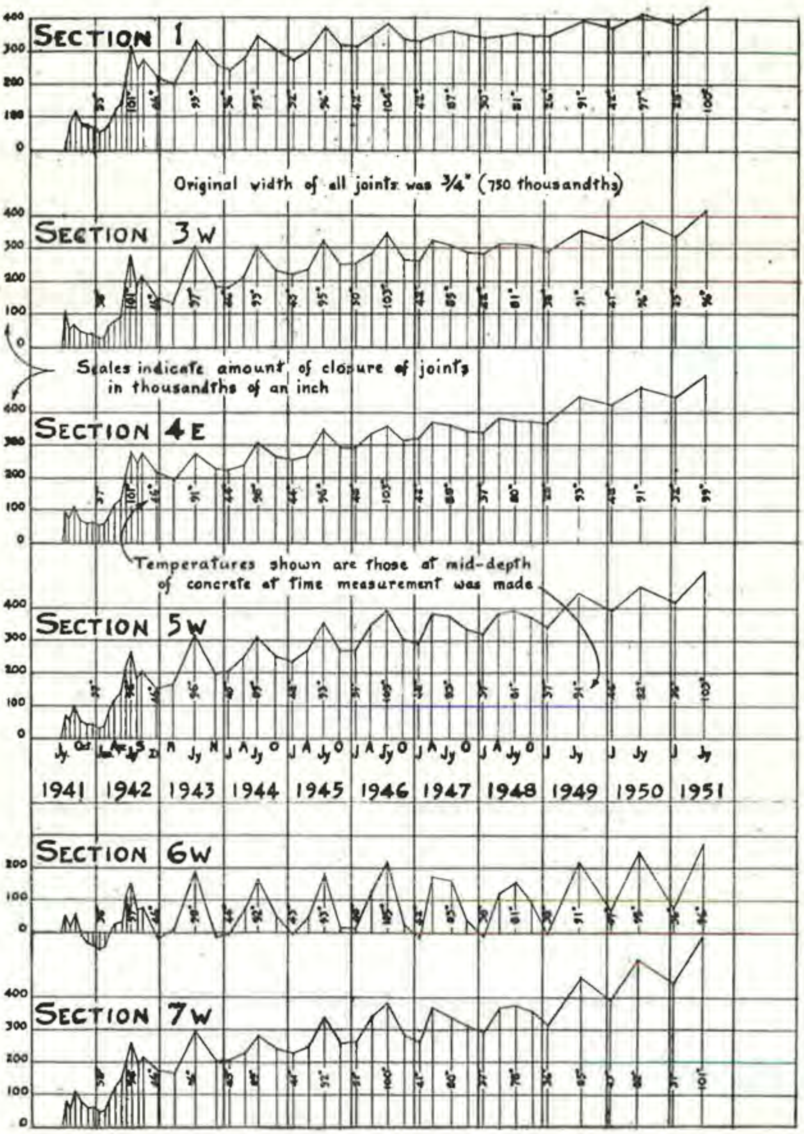


Figure 2. Lombard-Killingsworth pavement. Closure of expansion joints.

sections. Each contraction joint, by its failure to completely close during high temperatures, adds to the expansion joint closure. The seasonal movement after the first few years is principally affected by the action of the end slabs, which are 60 feet in length for the reinforced section and only 15 feet in length for the unreinforced sections.

It was expected that the subgrade drag would prevent the opening of the contraction joints in the central portion of the mile-long section. This effect has been relatively slight. Figure No. 3 shows the openings of the contraction joints in the five unreinforced test sections at the winter measurement when the joints are in their widest open position. The measurements shown are the average of the 1948 to 1950 readings. It will be noted that except for the end joints the openings of the joints in the mile-long section are approximately uniform. The opening of the joints in it, however, are much less than in the shorter sections.

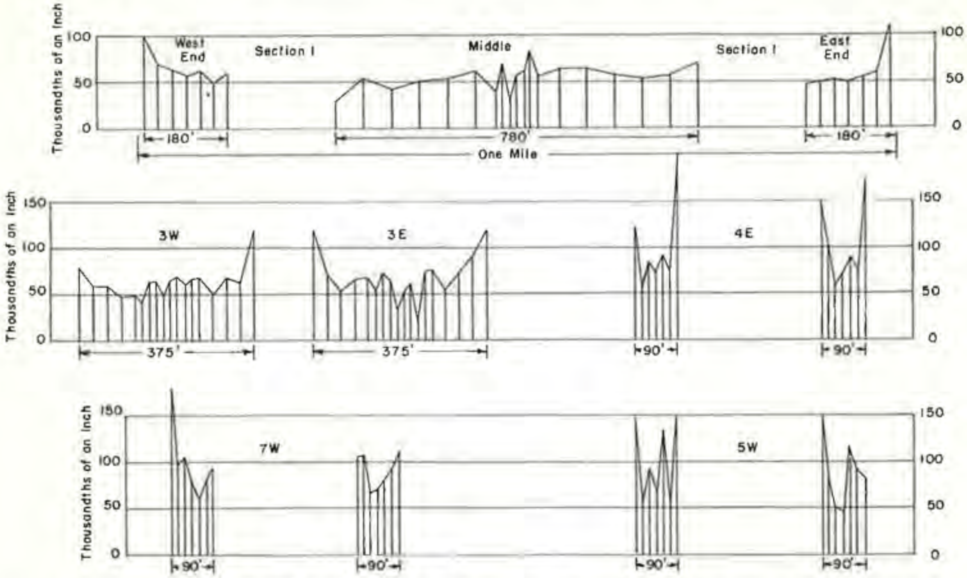


Figure 3. Lombard-Killingsworth pavement. Relative openings of contraction joints. Plotted values show the opening in thousandths of an inch of each contraction joint, as indicated by the mean of the three winter readings of 1948, 1949 and 1950, Average temperature of concrete at the times measurements were made, 37° F.

In Figure No. 4 are shown the average width of openings of the contraction joints in each of the six sections. The width of opening in the five unreinforced sections has remained practically constant since 1946. The width is less in the long sections, Sections No. 1 and No. 3, than in the 120-foot sections.

The movement at the single contraction joint at the mid-point of the 120-foot subsections of Section 6 is of considerable interest. The graph in Figure No. 4 shows the average movement of the five contraction joints measured in this section. The opening of these joints is approximately double that of the joints between the slabs of the other sections having 15-foot spacing of contraction joints. The seasonal movement between summer and winter conditions is also about double the movement between the 15-foot slabs. The joint widths are still increasing after ten years of service.

It will be noted that the openings of contraction joints during cold weather are in the range of 0.07 inch in Sections 1 and 3 and in general approach 0.10 inch in the 120-foot sections. In Section 6 with the 60-foot spacing between contraction joints the openings in the winter season are approximately 0.20, and even in summer the openings are more than 0.10. Sutherland and Cashell in their work on the "Structural Efficiency of Transverse Weakened-Plane Joints"[4] found that, "It must be concluded that aggregate interlock cannot be depended upon to give effective stress control throughout the full yearly temperature cycle in pavements with contraction joint spacings such that the joints can open an amount greater than approximately. 0.04 inch. The contraction joint measurements on this Oregon project show that even in the middle of the mile-long section the openings greatly

[4] "Structural Efficiency of Transverse Weakened-Plane Joints," Highway Research Board Report 3B, 1945.

exceed this figure. The almost complete absence of faulting and pumping and the general excellence of the pavement must be attributed to other causes than load transference across the joints, probably to the excellent base of gravel under the slabs.

It is probable that the width of contraction joint openings in the Oregon project is not typical of pavements in general. It will be noted later in this report that a slight shrinkage of the concrete has taken place rather than a growth as is usual in most concretes. Apparently the openings of the joints in the central portions of the larger sections is due to this shrinkage rather than to movement along the subgrade. It is

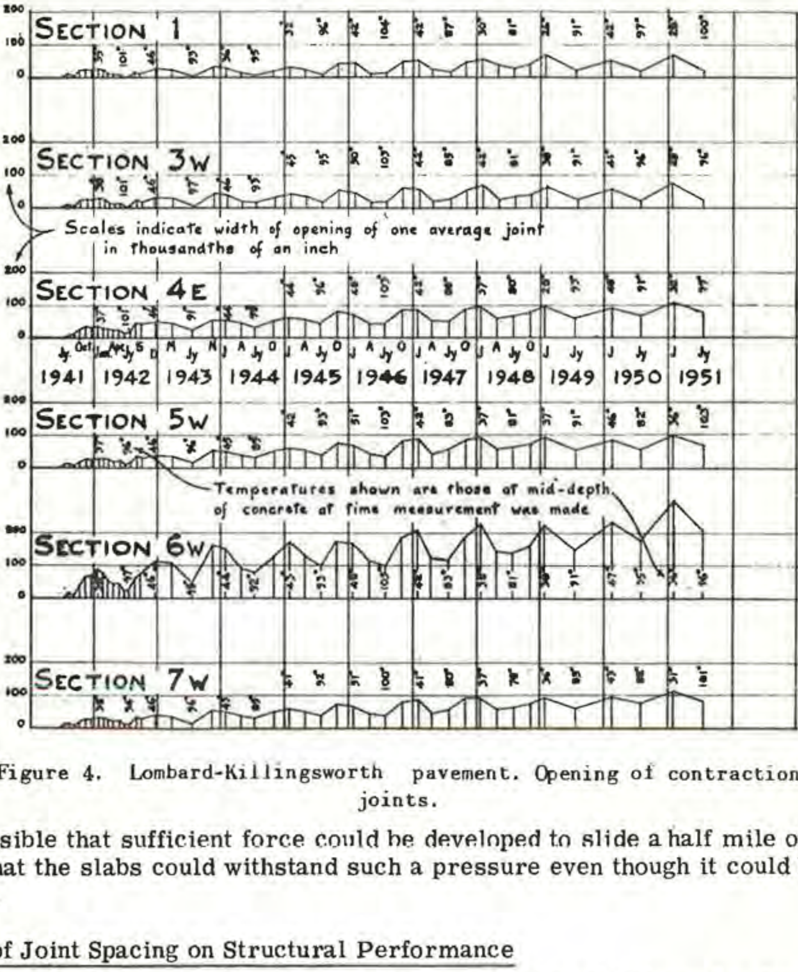


Figure 4. Lombard-Killingsworth pavement. Opening of contraction joints.

hardly possible that sufficient force could be developed to slide a half mile of pavement slab nor that the slabs could withstand such a pressure even though it could have been developed.

Influence of Joint Spacing on Structural Performance

There is no evidence that the joint spacing has had any effect on the structural performance or the riding quality of the pavement. As stated earlier, all of the sections are in such excellent condition that no significant differences can be detected. In 1941, 1944 and again in 1950, a graphic record of surface irregularity was made. These records were made on the same sections each year and as nearly as possible along the same line. A print of the three records is shown in Figure No. 6. There seems to be no significant change from 1944 to 1950. In each of these two later records the effect of the joints is more pronounced than in the 1941 record.

Faulting and Pumping

There has been no pumping on this project. Bronze pins were set in the pavement slabs on each side of a large number of expansion joints and contraction joints. The elevations of these points were determined at the time the pavement was opened to traffic and again in 1950. The greatest settlement observed at any point was 0.039 foot. The greatest differential movement between two adjacent points was 0.014 foot. This happened at three points at a contraction joint in Section No.5 where dowels were placed across the crack and at an expansion joint and a contraction joint in Section No.7 where no dowels were used. The average and maximum differential movement at all expansion and contraction joints is shown in Table 4. Neither the length of section between expansion joints nor the use of dowels seems to have had

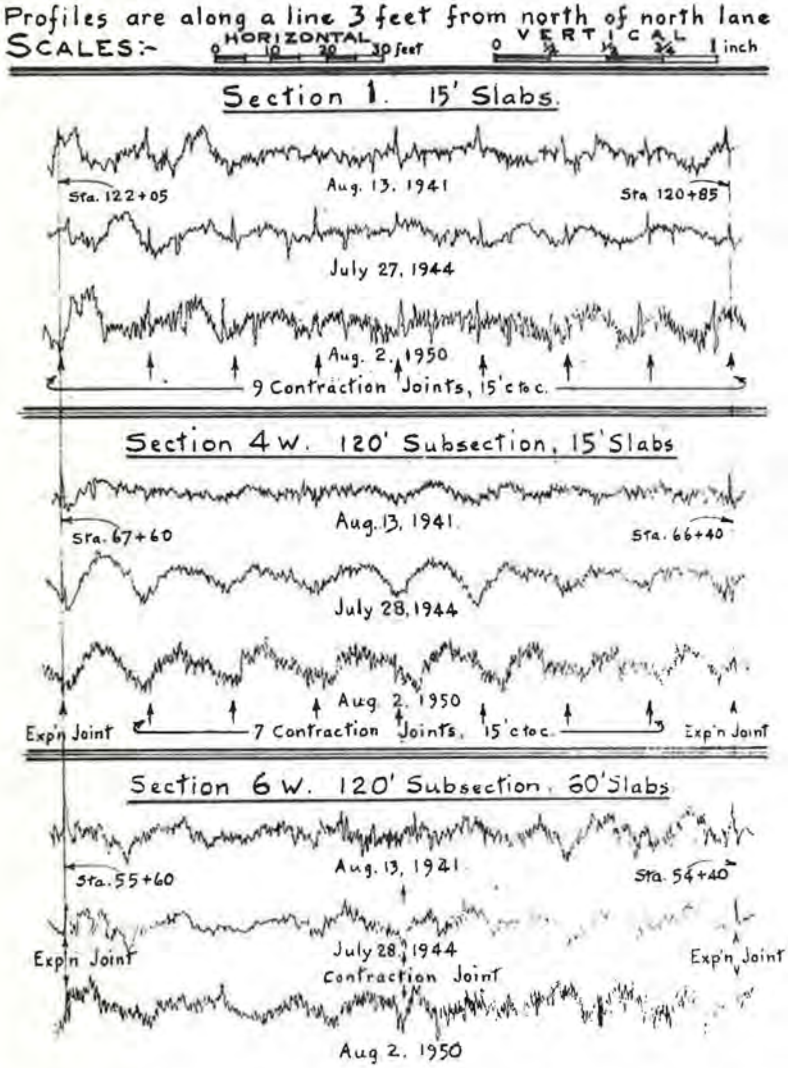


Figure 6. Lombard-Killingsworth pavement, Portland. Changes in surface irregularity at 3 and 9 years after construction.

any significant effect on the differential movement. The gravel base under the pavement probably accounts for the general excellence.

Relative Performance of Plain and Reinforced Concrete

This project did not give data that would warrant drawing conclusions as to the relative performance of the plain and reinforced sections. All sections are in approximately equal condition at the end of ten years of service. The closure of the

TABLE 4

DIFFERENTIAL MOVEMENT AT JOINTS

Section No.	Joint Type	Average	Maximum	Load Transfer
1	Contraction	0.005	0.012	No Dowels
3	Contraction	0.005	0.011	No Dowels
3	Expansion	0.003	0.005	Dowels
4	Contraction	0.005	0.010	No Dowels
4	Expansion	0.002	0.004	Dowels
5	Contraction	0.004	0.014	Dowels
5	Expansion	0.004	0.008	Dowels
6	Contraction	0.005	0.009	Dowels
6	Expansion	0.004	0.008	Dowels
7	Contraction	0.006	0.014	No Dowels
7	Expansion	0.007	0.014	No Dowels

expansion joints is less, and the opening of the contraction joints is greater in the reinforced sections than in the plain concrete sections. These differences have not appreciably affected the performance of the pavements. If any inference can be drawn it is that with a good subsoil and an adequate granular base reinforcing of concrete pavements is not necessary.

Structural Condition of Slab Corners

There have been no corner breaks, as the term is ordinarily used, in the Oregon project. In the Highway Research Report 3B, attention was called to the surface spalling at the corners due to faulty installation of the asphaltic filler in the contraction joints. This took place in the first few years of service and has not progressed further. The contraction joints were made by inserting a one-fourth inch by 2-inch strip of asphalt-impregnated felt into the upper surface of the pavement. This was

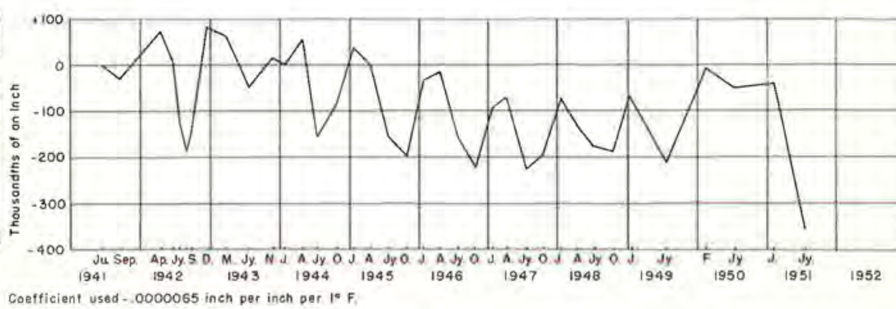


Figure 5. Lombard-Killingsworth pavement. Length changes in 120 feet corrected for a concrete temp. of 40° F.

done before the passage of the finishing machine, and in some cases the felt was pulled away from the side forms several inches leaving a small area at the edge of the pavement in direct bearing. As the slabs curled due to temperature change, these areas took the entire pressure and spalled for the depth of the felt strip. This is a surface defect and not a corner break due to load. It occurs in all sections regardless of joint spacing or load transfer devices.

Evidence of Slab Growth

There is apparently no slab growth in the sense of a swell of the concrete due to physical or chemical change in the concrete itself. There is some slight evidence that shrinkage has been taking place. The procedure used in measuring actual change in the length of concrete slabs does not allow for the separation of the length changes due to temperature, moisture content, elasticity and other external causes, from the internal changes usually classified as growth or shrinkage. Records of the gross length change in all sections have been kept. There is a remarkable similarity in the records for all sections. Figure No. 5 is the record for Section 7. The net change in length is the difference between the outward movement of the ends of the section and the contraction joint openings. In this figure the length changes have been corrected for temperature change. No correction has been attempted for other external causes. A bar graph showing rainfall is included in the figure. The change in net length follows the rainfall, being less in winter than in summer. In general the slope of the length change is down, indicating a slight tendency toward shrinkage.

SUMMARY

As an experimental project, the Lombard Street-Killingsworth Street Section has been a disappointment. The excellent quality of the soil and the adequate granular base have masked any differences in performance of the several subsections. All subsections, regardless of joint spacing, load transfer or other variables, have withstood ten years of heavy traffic without noticeable deterioration. A few conclusions can be drawn which are certainly applicable to this project and which can be extended to similar projects.

Expansion joints can be eliminated in pavements built from sound materials. There is no indication of any kind that a mile-long section without expansion joints has been injured in any way. When expansion joints are placed, they begin to close immediately after construction, and this closure has continued at a fairly even rate for ten years. It is probable that eventually expansion and contraction joints will be of approximately equal width.

All contraction joints, even in the mile-long section where restraint is a maximum, have opened enough so that aggregate interlock across the joints is not effective. With a yielding base, some load transfer device is probably advisable. In this particular project the quality of the base and subgrade is such that load transfer is apparently not necessary.

Even though the measurements show that the contraction joints cannot develop aggregate interlock, there is no indication of pumping or faulting at the joints. The traffic volume is great enough to have produced these effects if they are to occur. Their absence is undoubtedly due to the excellent subgrade and the adequate granular base. The durability of the pavement is evidence of the advisability of the use of granular bases where such use is economically feasible.