Report on Experimental Project in Minnesota

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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			PIT Nº 2		
		MAX.	MIN.	AV.	MAX.	MIN.	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

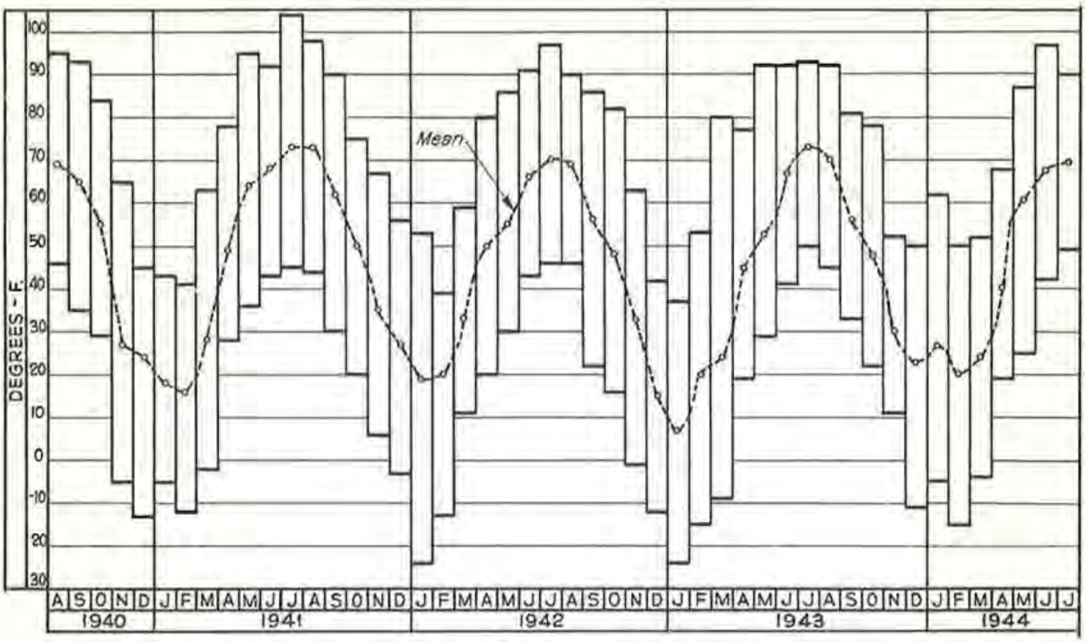


Figure 1. Air temperature data.

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

		Ig. Loss	CaO	MgO	Fe_2O_3	Al_2O_3	SO_3	SiO_2	Free CaO	TOTAL
CHEMICAL ANALYSIS	ANALYSIS BY WT. - %	0.93	64.4	1.85	2.94	5.91	1.47	21.35	0.93	99.78

		C_2S	C_3S	C_3A	C_4AF	$CaSO_4$	Free CaO	Free MgO	Free Mn_2O_3	Ig. Loss
CHEMICAL ANALYSIS	CALCULATED CHEMICAL COMPOSITION - % BY WT.	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 № 1	363.4	441.2	3:19	6:12
	44	8/20/40 - 9/20/40	Pit № 2	352.0	440.6	3:27	6:17

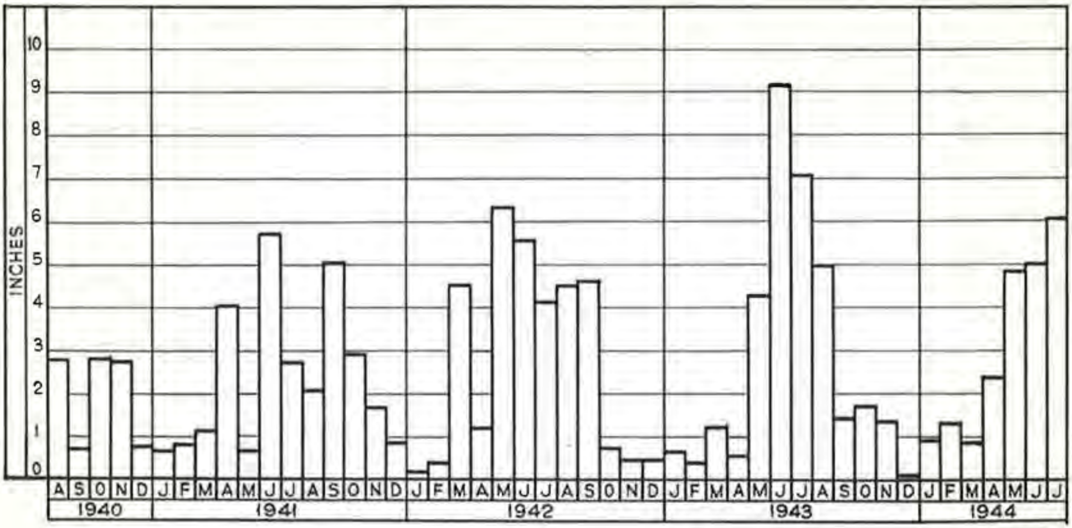


Figure 2. Precipitation data.

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

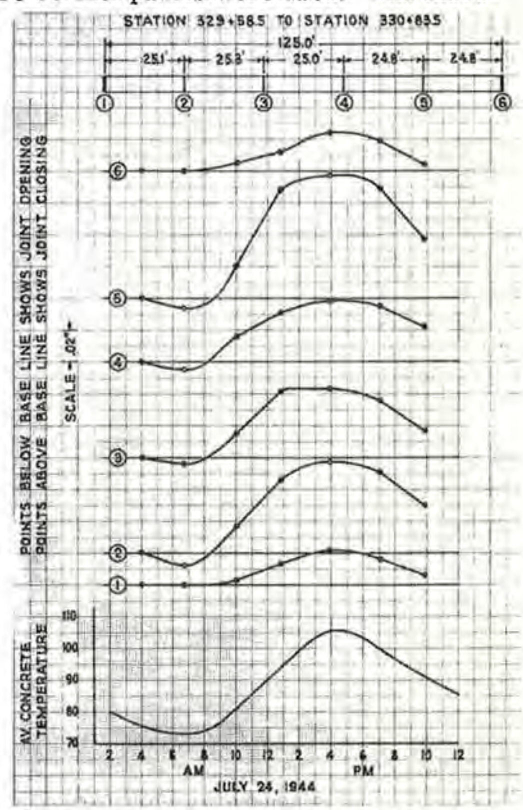


Figure 3. Daily changes in joint openings.

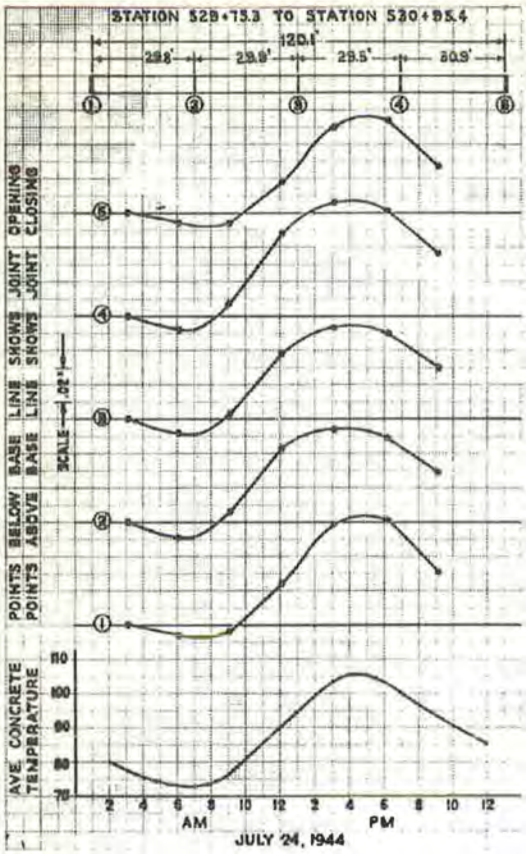


Figure 4. Daily changes in joint openings.

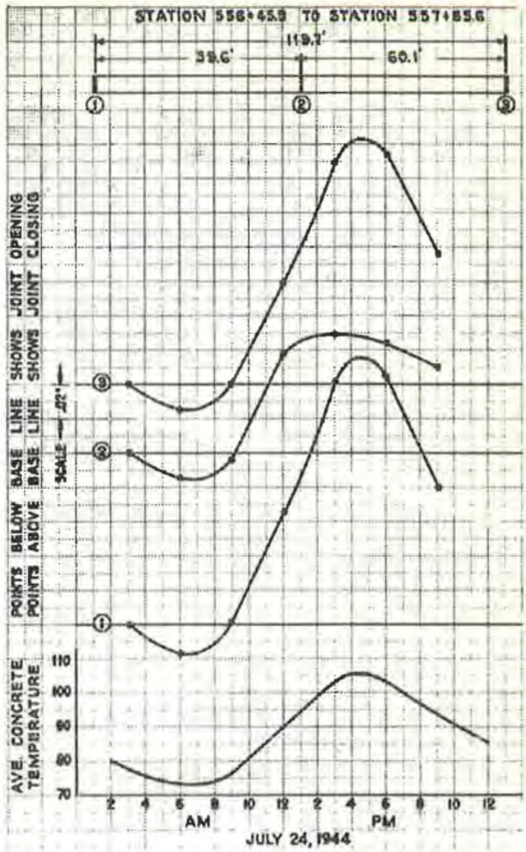


Figure 5. Daily changes in joint openings.

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 -

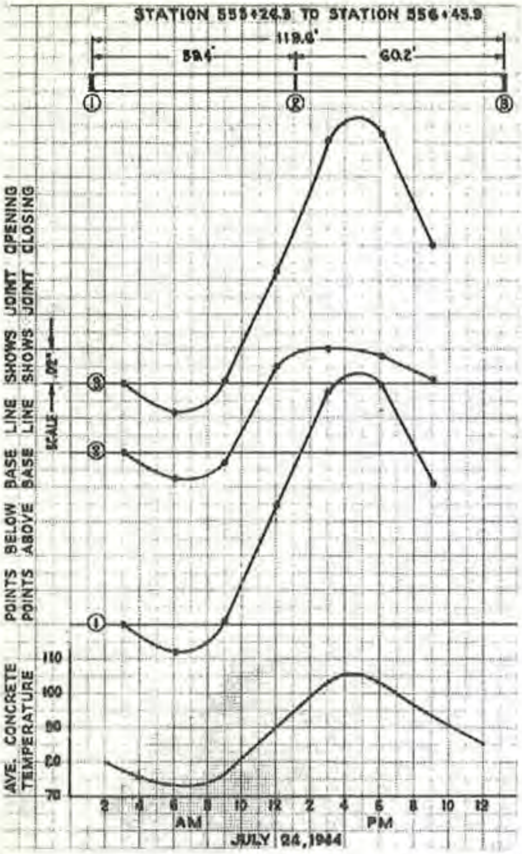


Figure 6. Daily changes in joint openings.

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

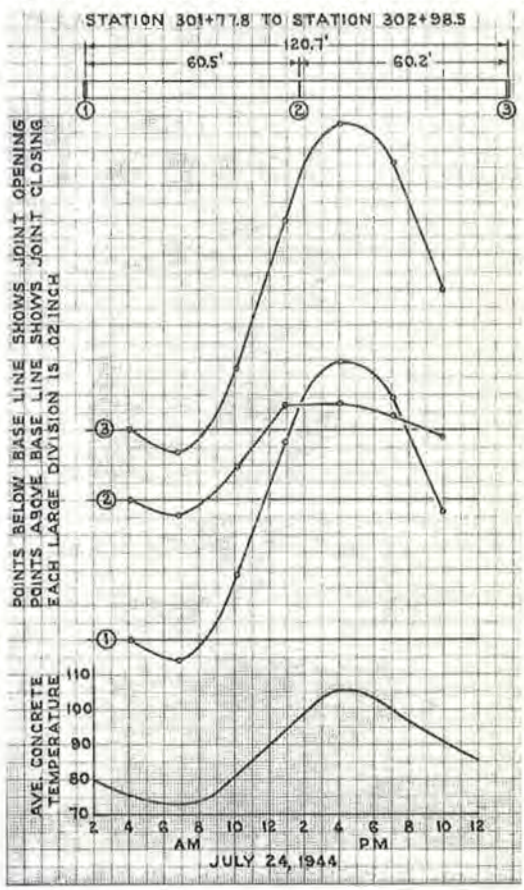


Figure 7. Daily changes in joint openings.

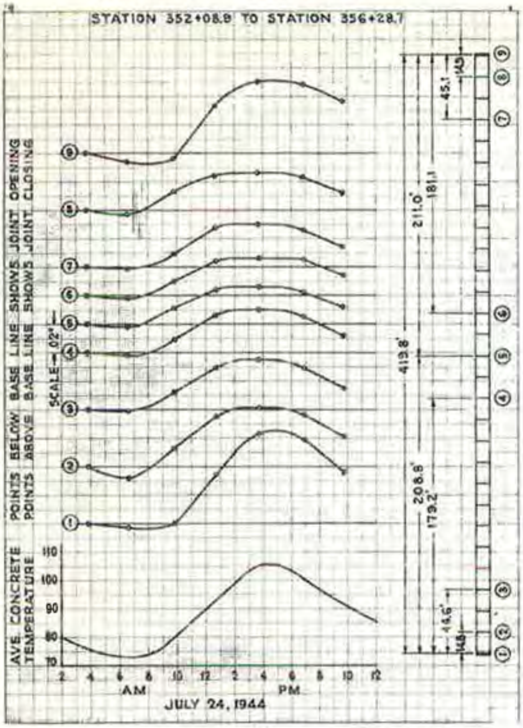


Figure 8. Daily changes in joint openings.

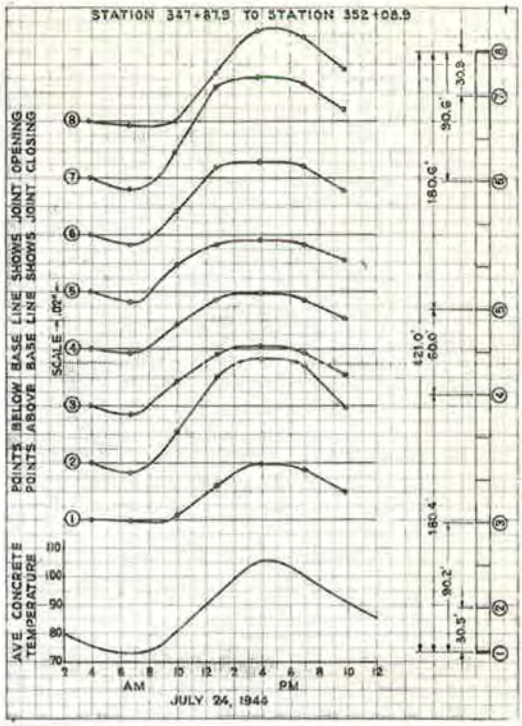


Figure 9. Daily changes in joint openings.

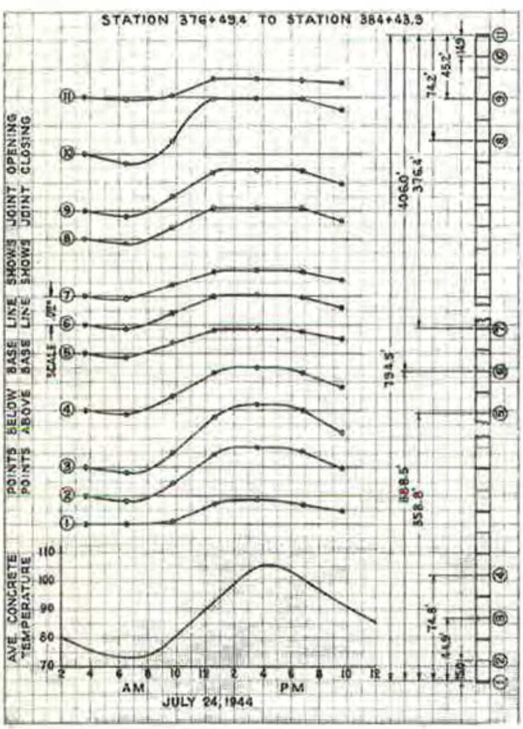


Figure 10. Daily changes in joint openings.

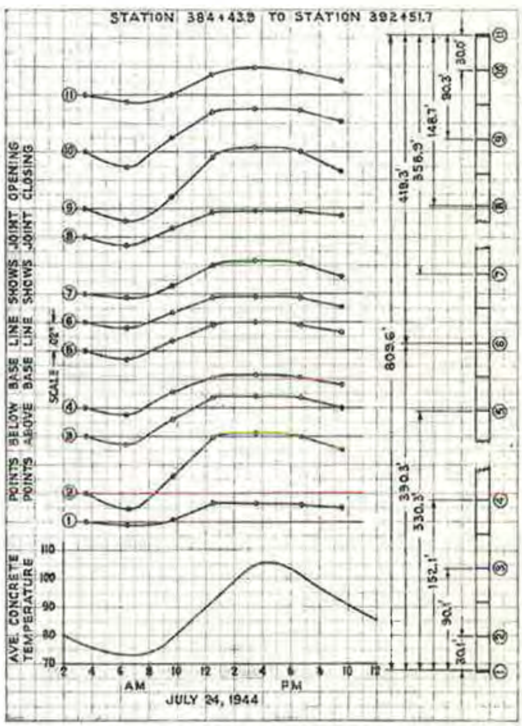


Figure 11. Daily changes in joint openings.

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.

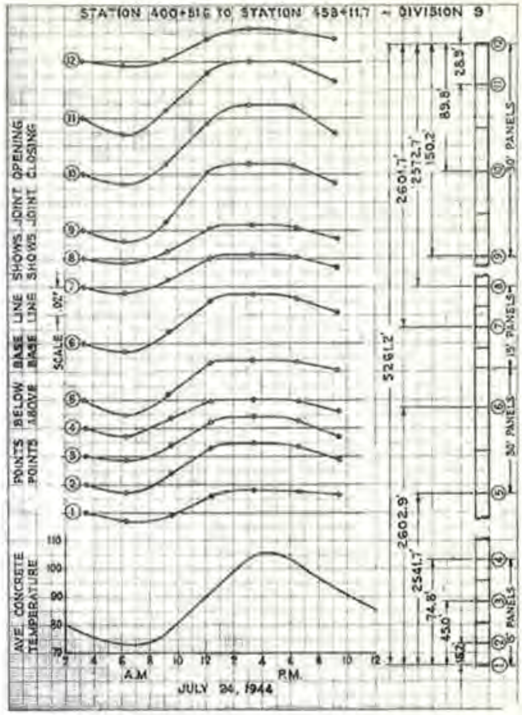


Figure 12. Daily changes in joint openings.

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.

TABLE 3

CONCRETE STRENGTH TESTS

Flex. Tests Made in Field (**)

STA. - STA.	Aggregates from Pit No. 1: 7 Day Tests: No. Tests	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)	14 Day Tests: No. Tests	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)	Aggregates from Pit No. 2: 7 Day Tests: No. Tests	Av. Unit Str.	Std. Dev. (*)	Coef. of Var. (*)	14 Day Tests: 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

STA. - STA.		Pavement Section	Source of Aggregates	Age of Cores (***): When Drilled	When Tested	Core Heights, 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 Strength (Corrected for H/d.): 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"	2.92%	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%

Core Heights: (Actual heights shown below are the averages of no. shown.)

(*) 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	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		90 DAYS		180 DAYS		1 YR.		2 YR.	
		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	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

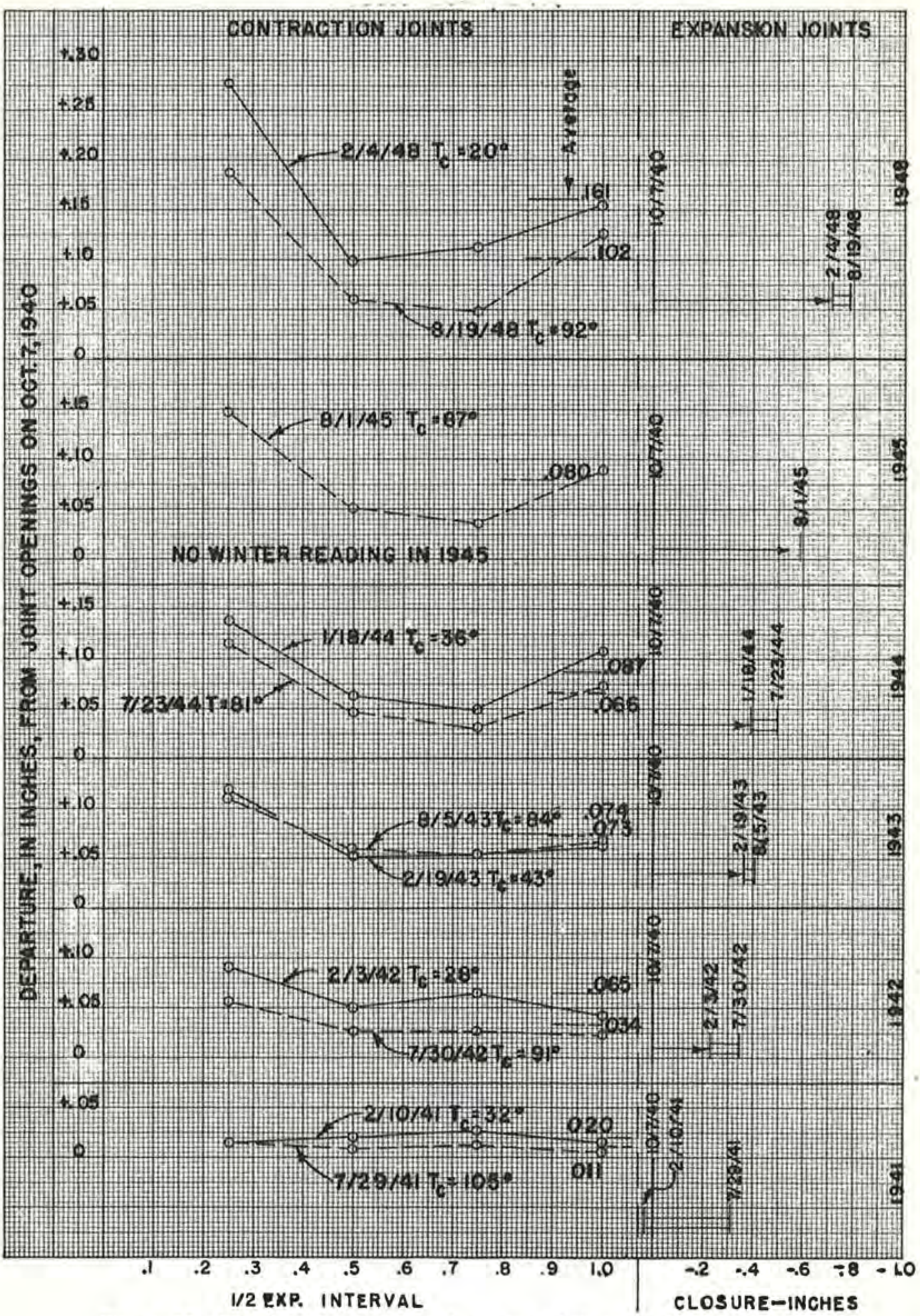


Figure 13. Annual and permanent changes in joint openings, expansion joint interval=120 feet, contraction joint interval=15 feet.

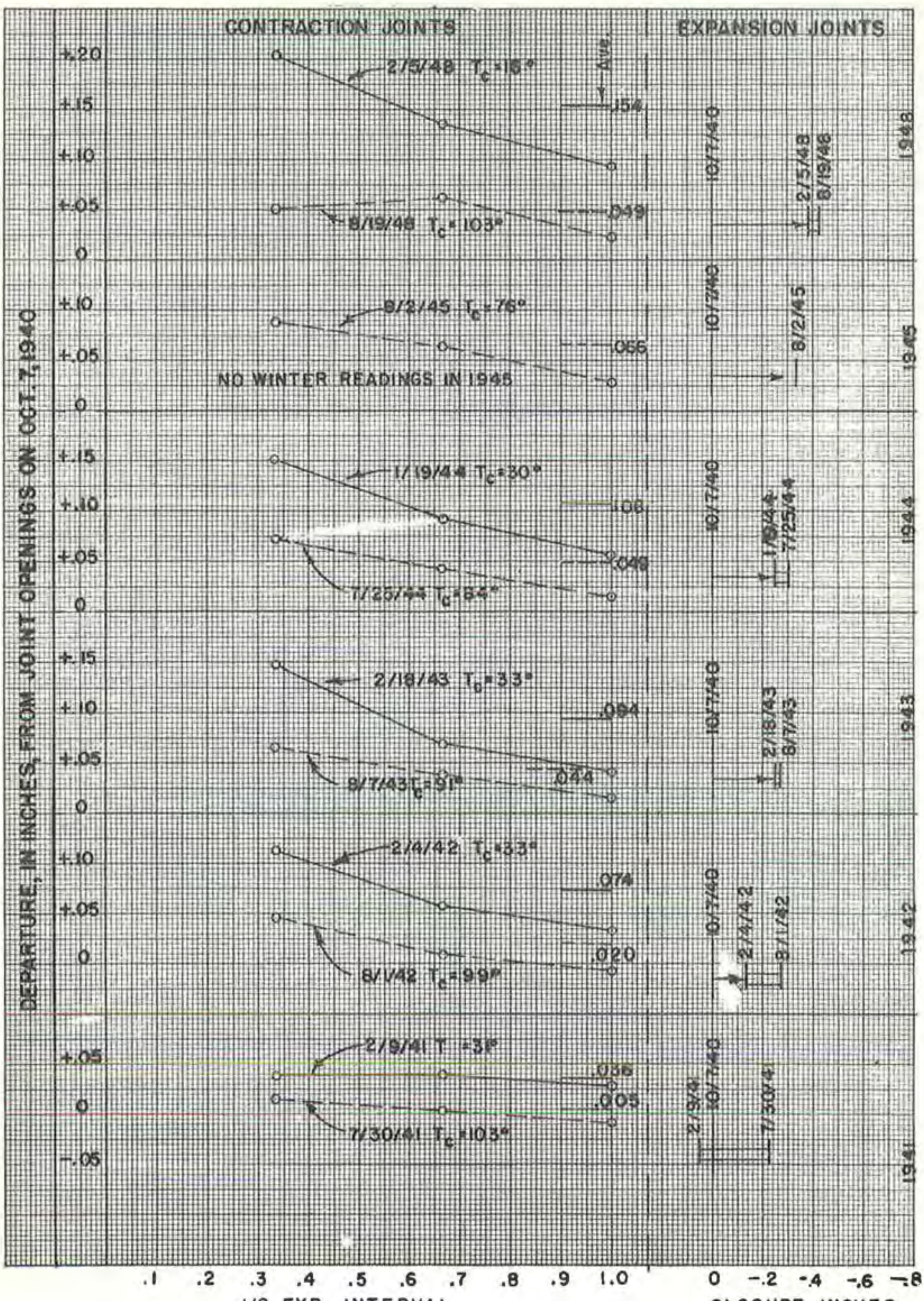


Figure 14. Annual and permanent changes in joint openings, expansion joint interval =120 feet, contraction joint interval=20 feet.

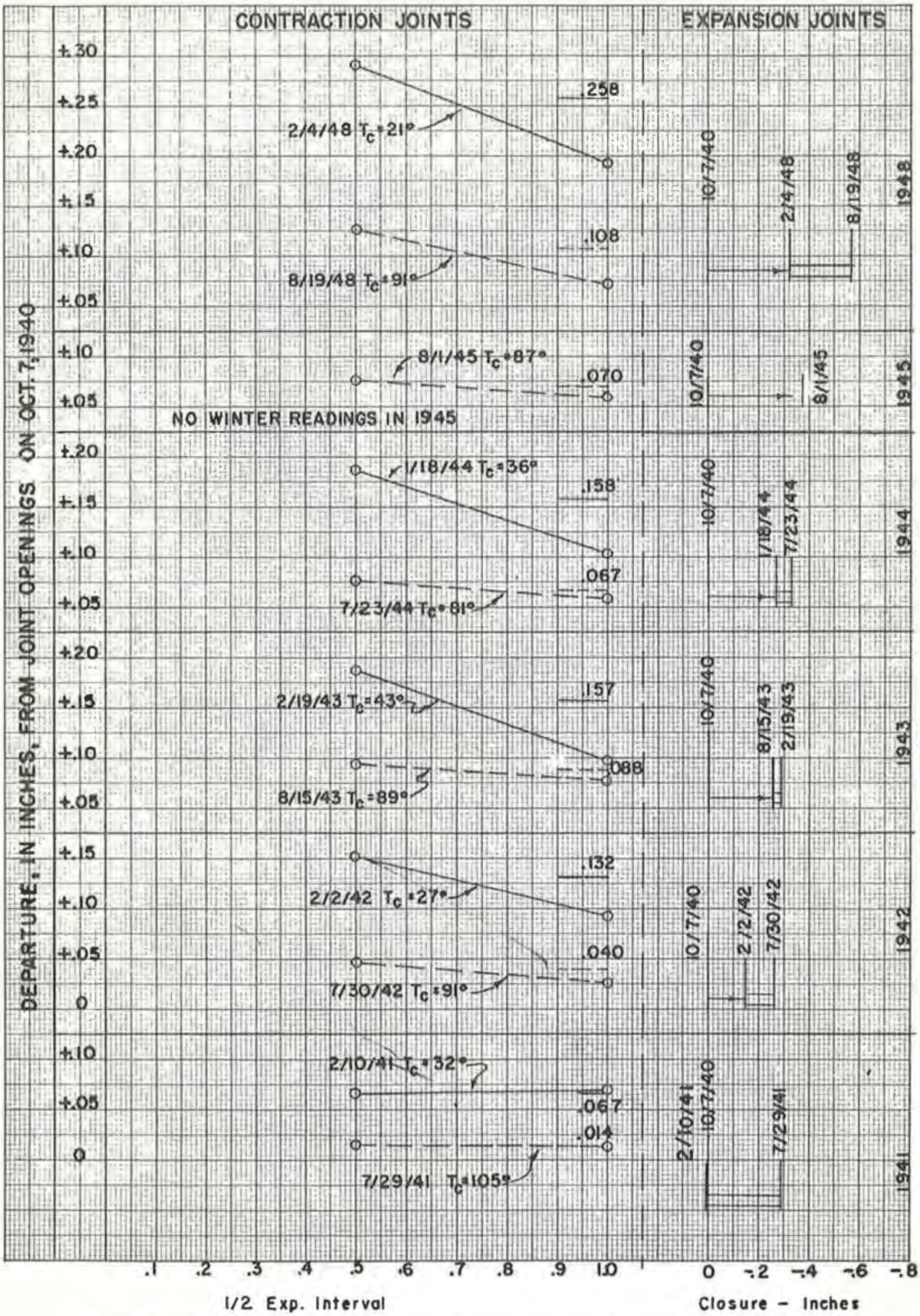


Figure 15. Annual and permanent changes in joint openings, expansion joint interval=120 feet, contraction joint interval=30 feet.

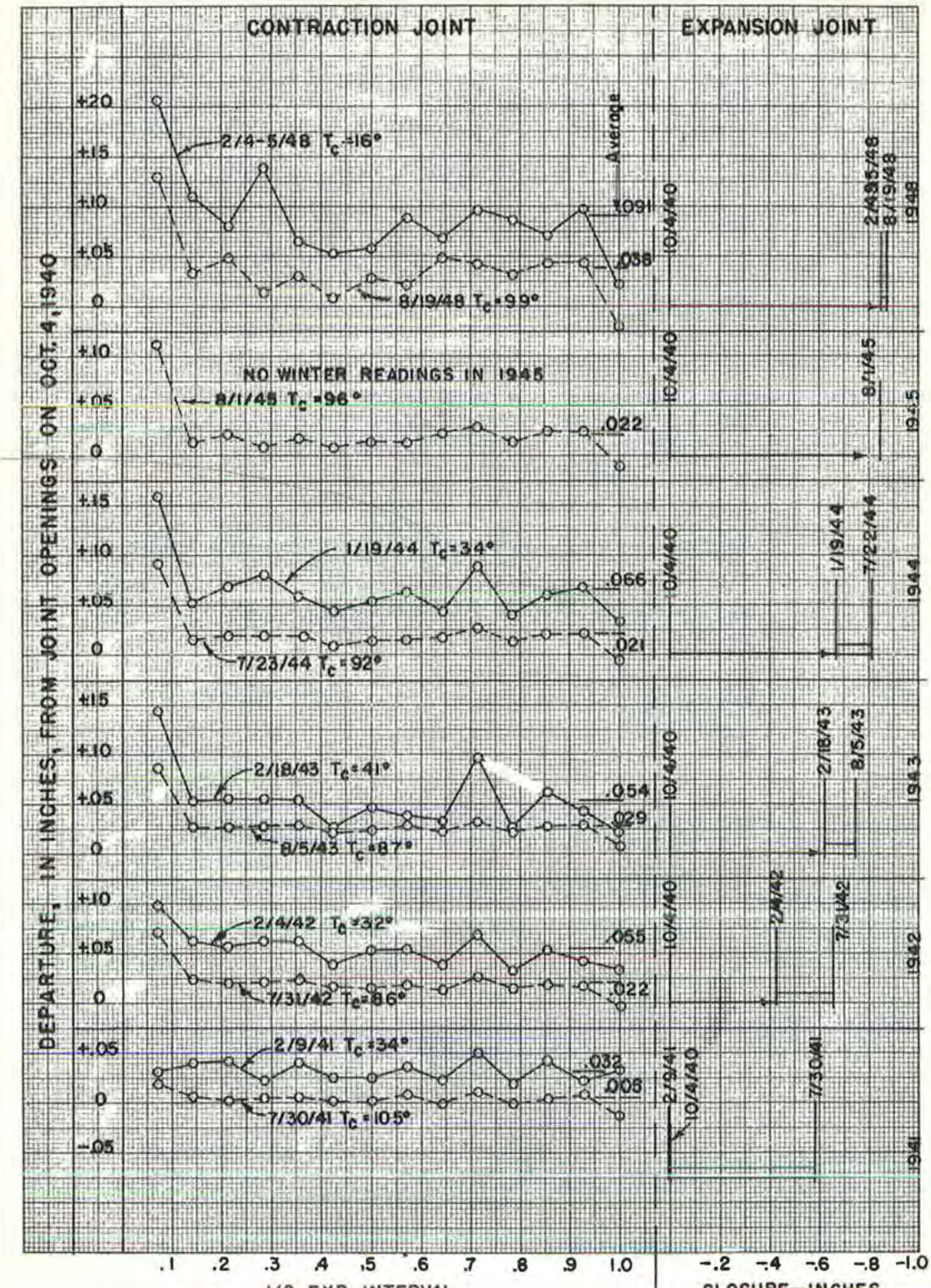


Figure 16. Annual and permanent changes in joint openings, expansion joint interval=420 feet, contraction joint interval=15 feet.

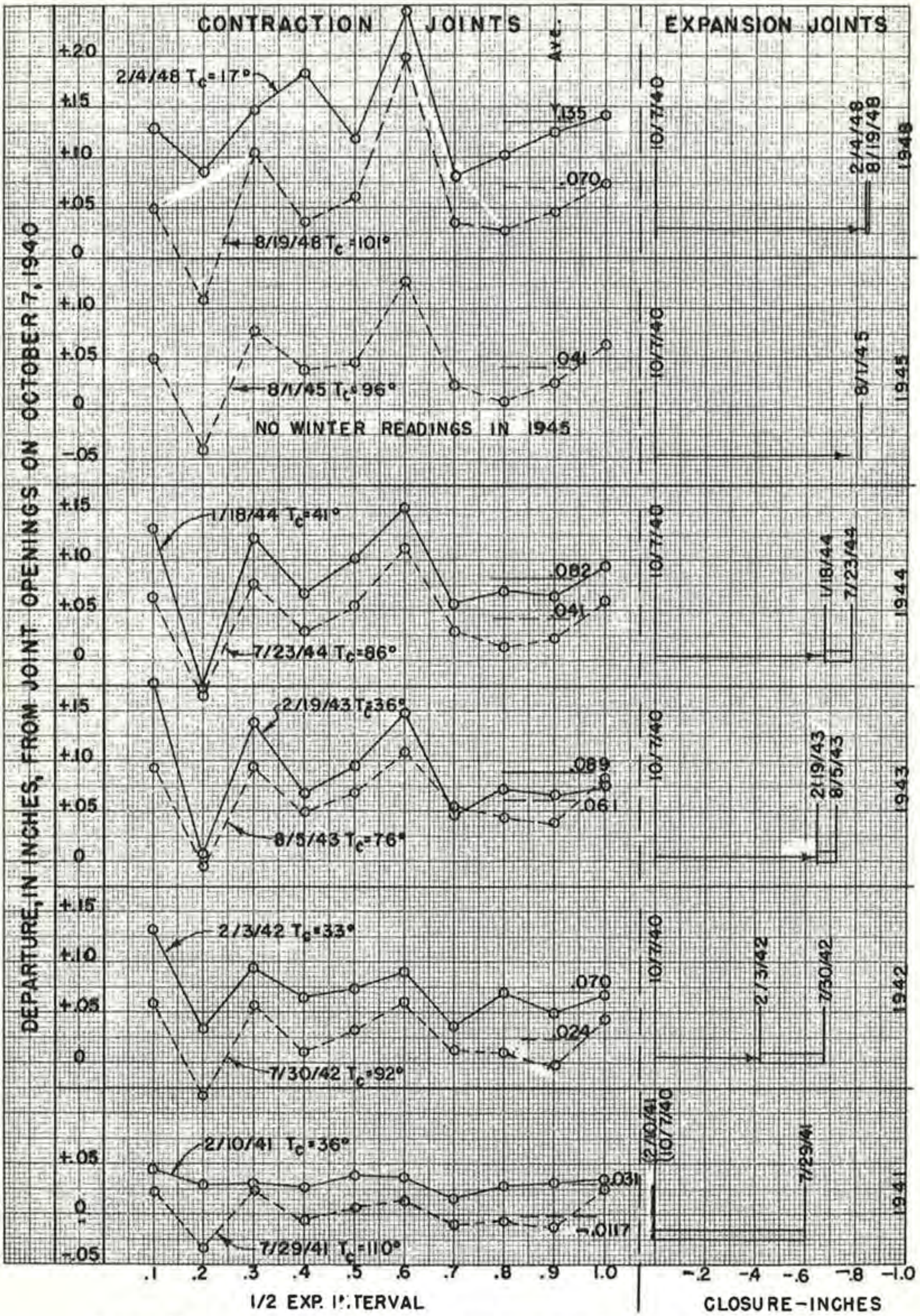


Figure 17. Annual and permanent changes in joint openings, expansion joint interval=400 feet, contraction joint interval=20 feet.

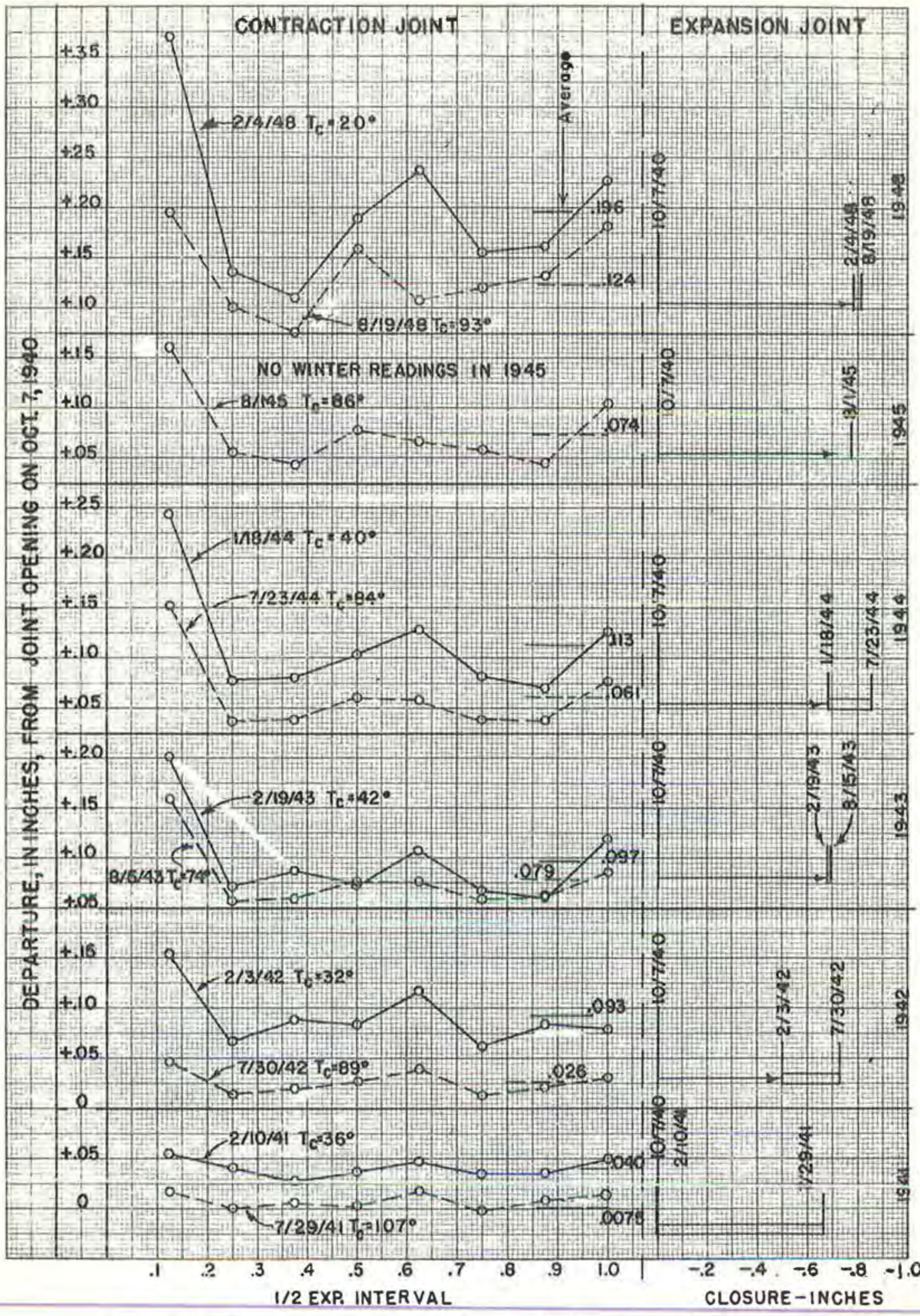


Figure 18. Annual and permanent changes in joint openings, expansion joint interval=400 feet, contraction joint interval=25 feet.

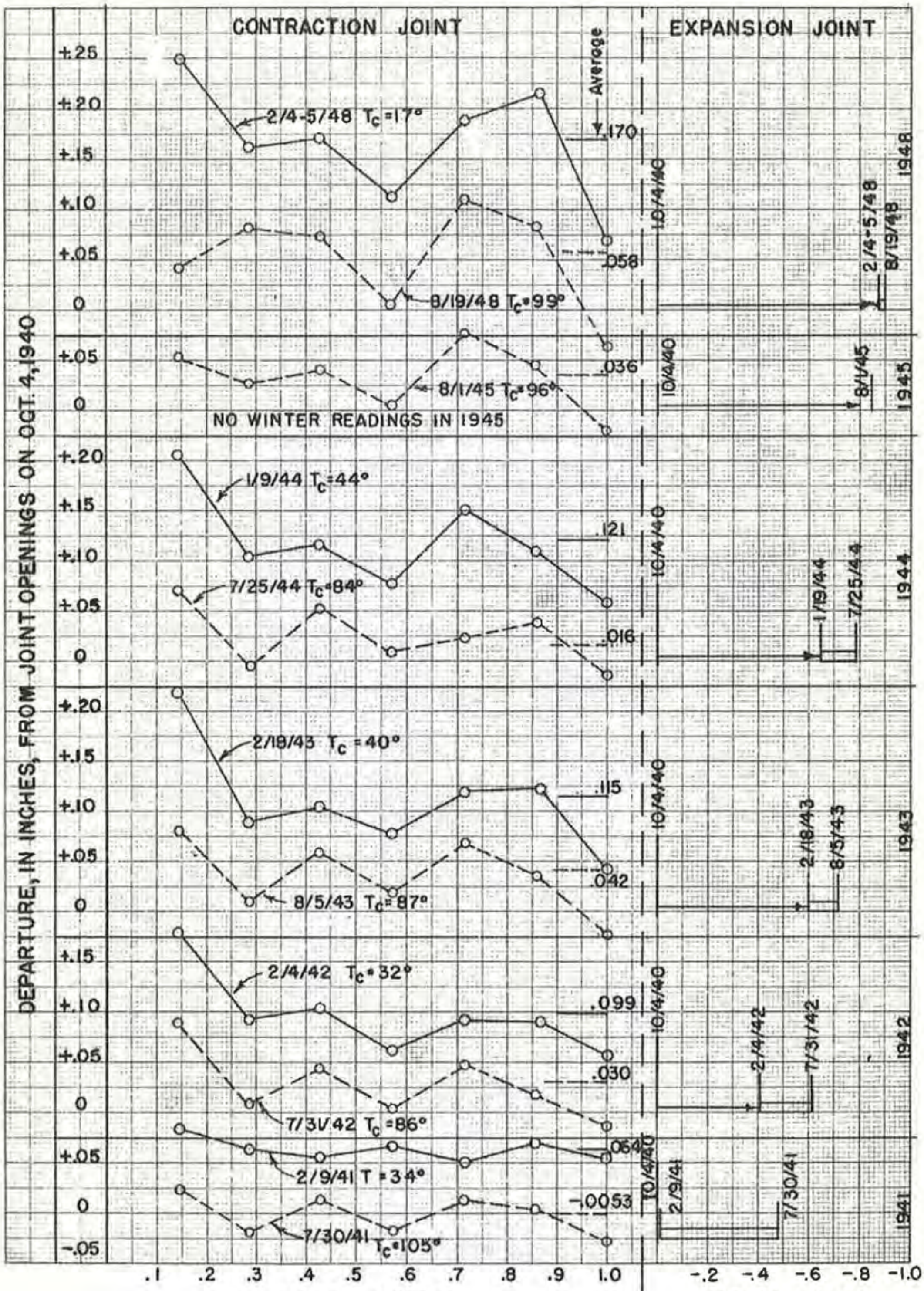


Figure 19. Annual and permanent changes in joint openings, expansion joint interval=420 feet, contraction joint interval=30 feet.

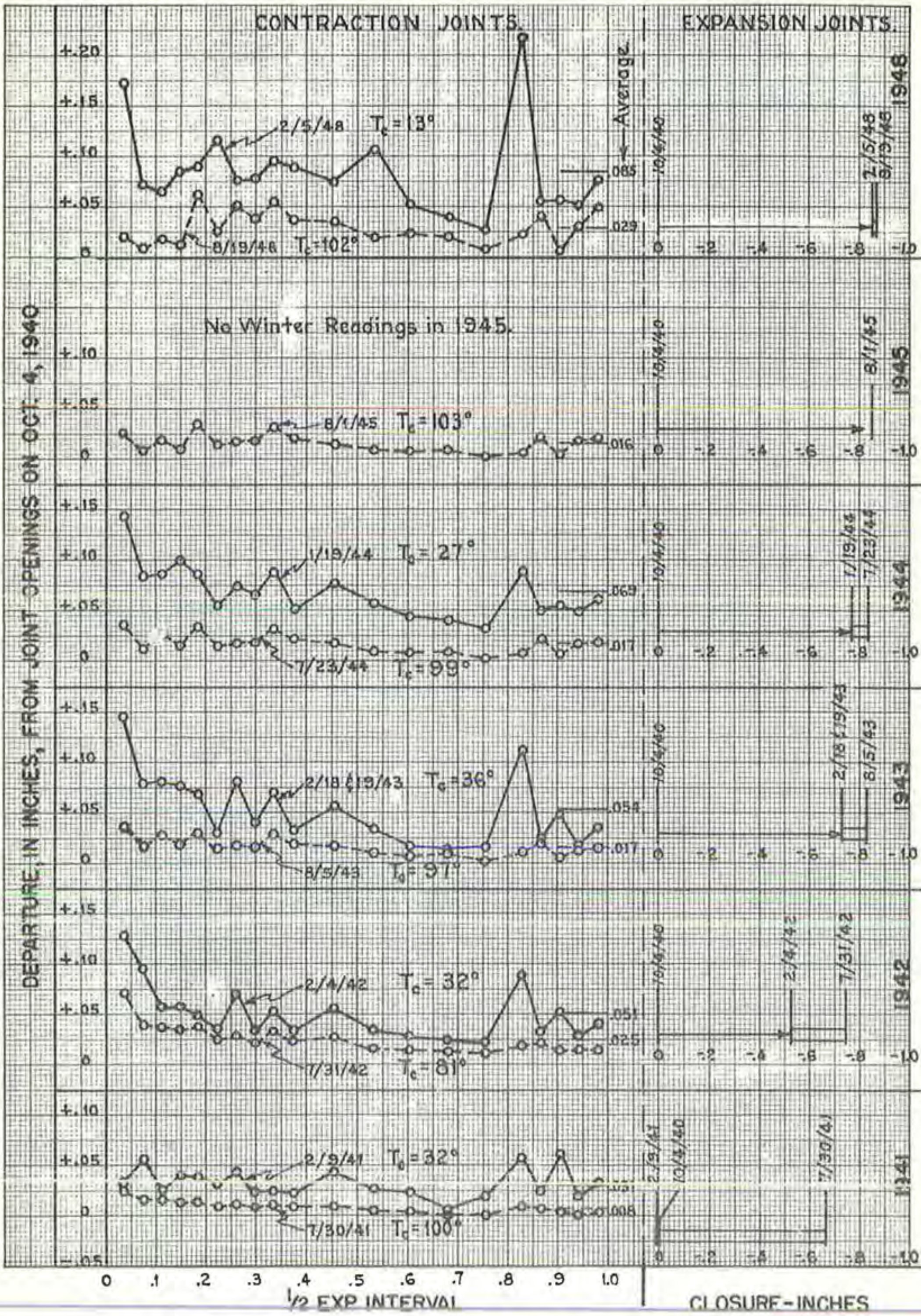


Figure 20. Annual and permanent changes in joint openings, expansion joint interval=795 feet, contraction joint interval=15 feet.

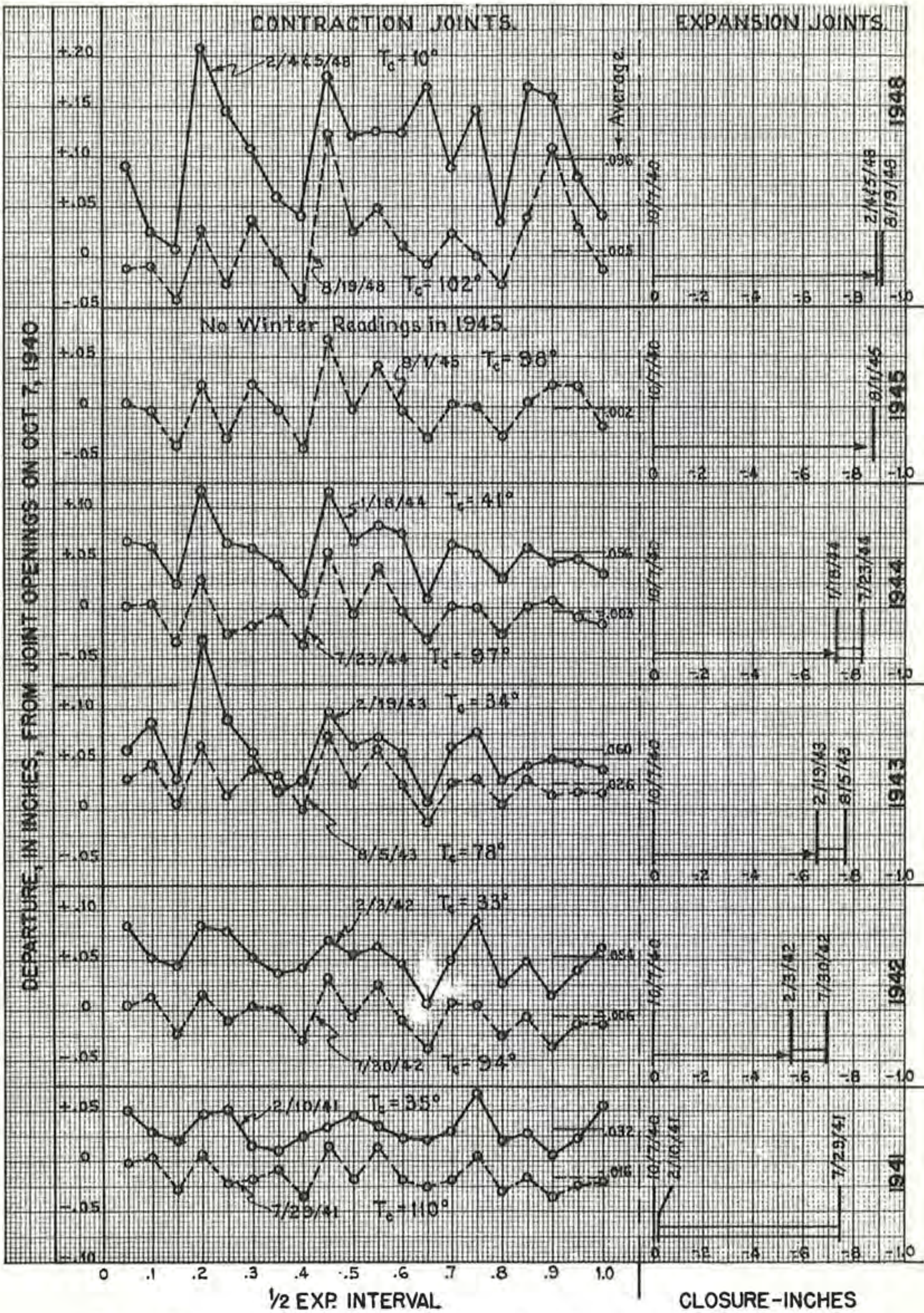


Figure 21. Annual and permanent changes in joint openings, expansion joint interval=800 feet, contraction joint interval=20 feet.

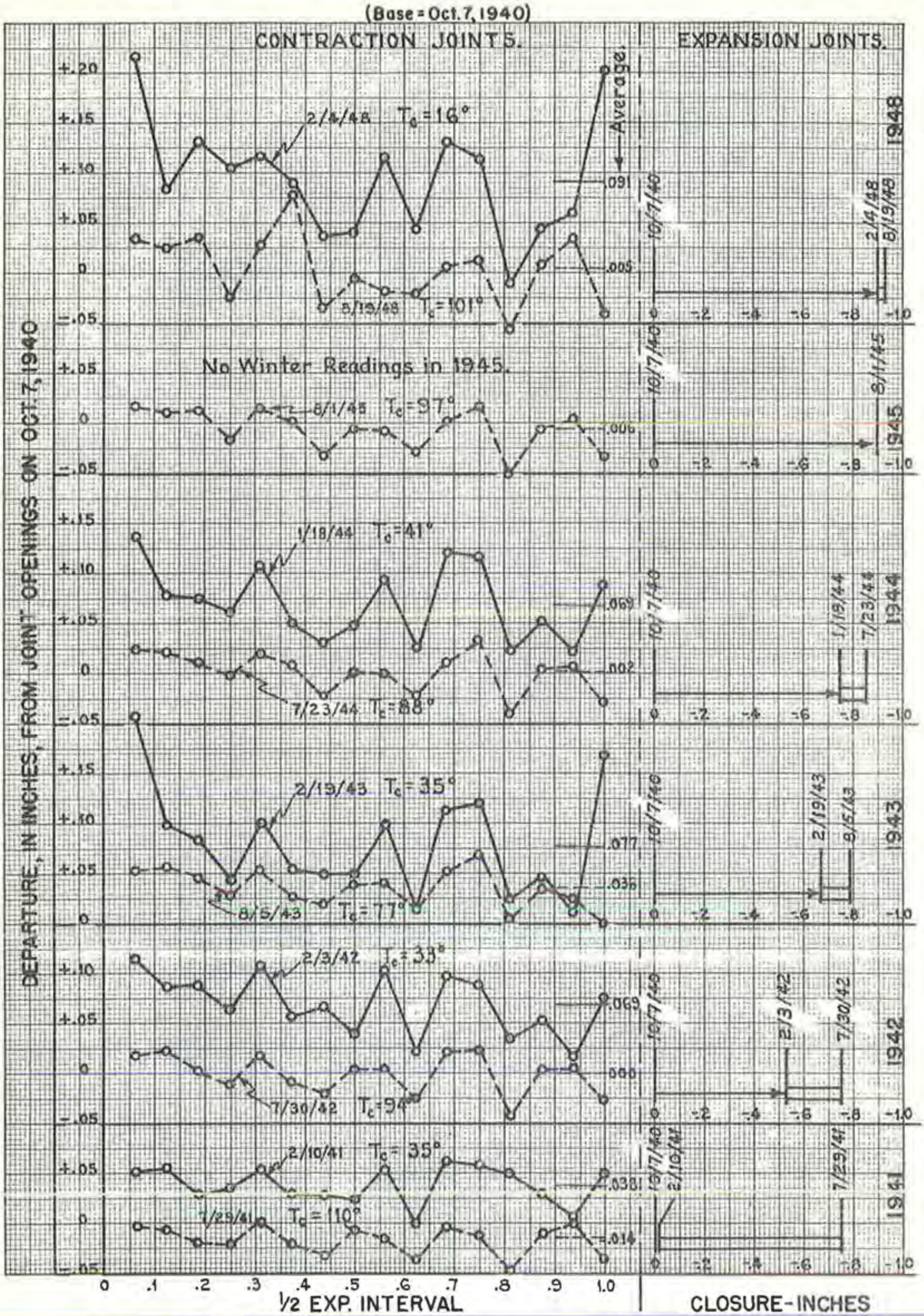


Figure 22. Annual and permanent changes in joint openings, expansion joint interval=800 feet, contraction joint interval=25 feet.

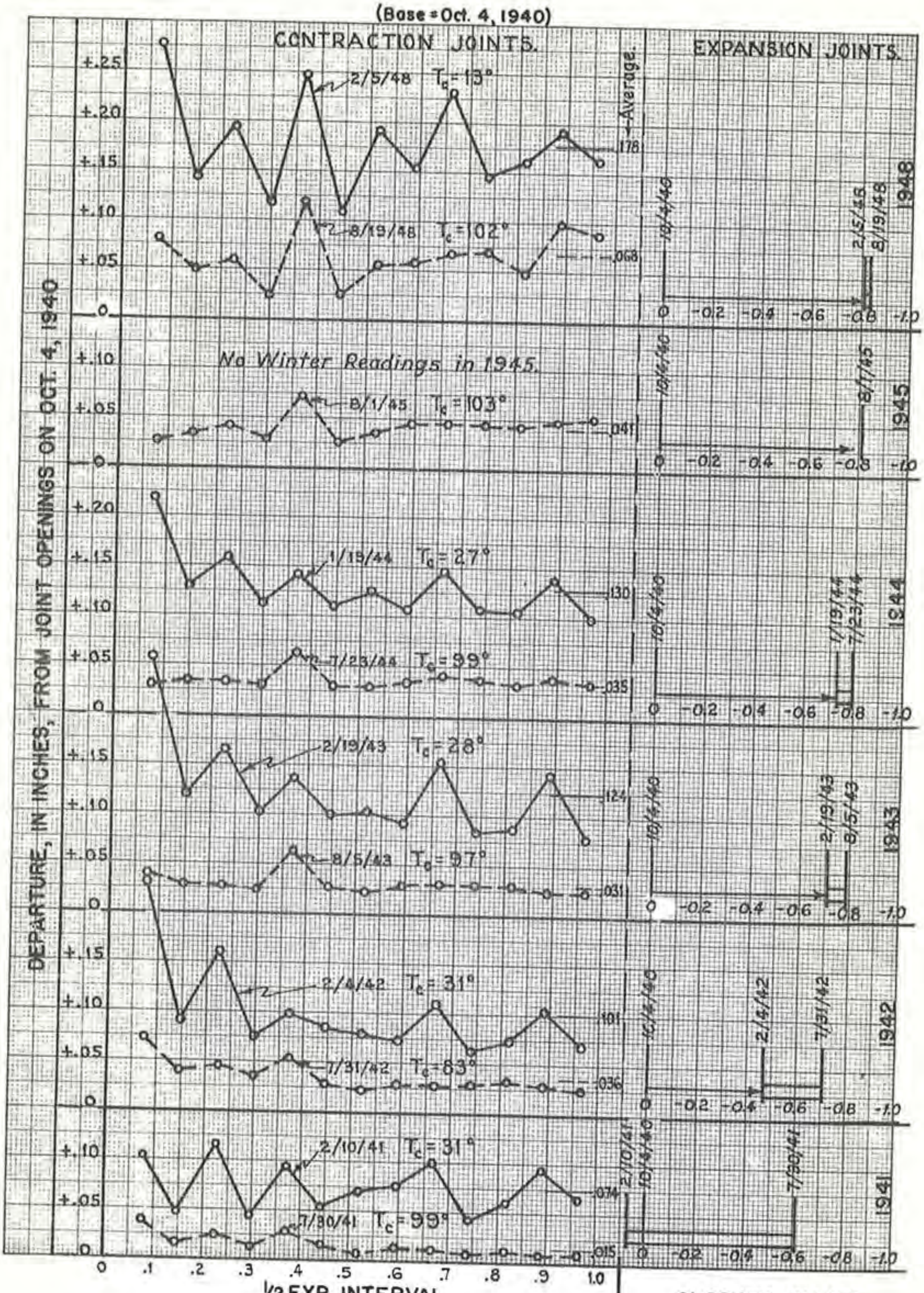


Figure 23. Annual and permanent changes in joint openings, expansion joint interval=810 feet, contraction joint interval=30 feet.

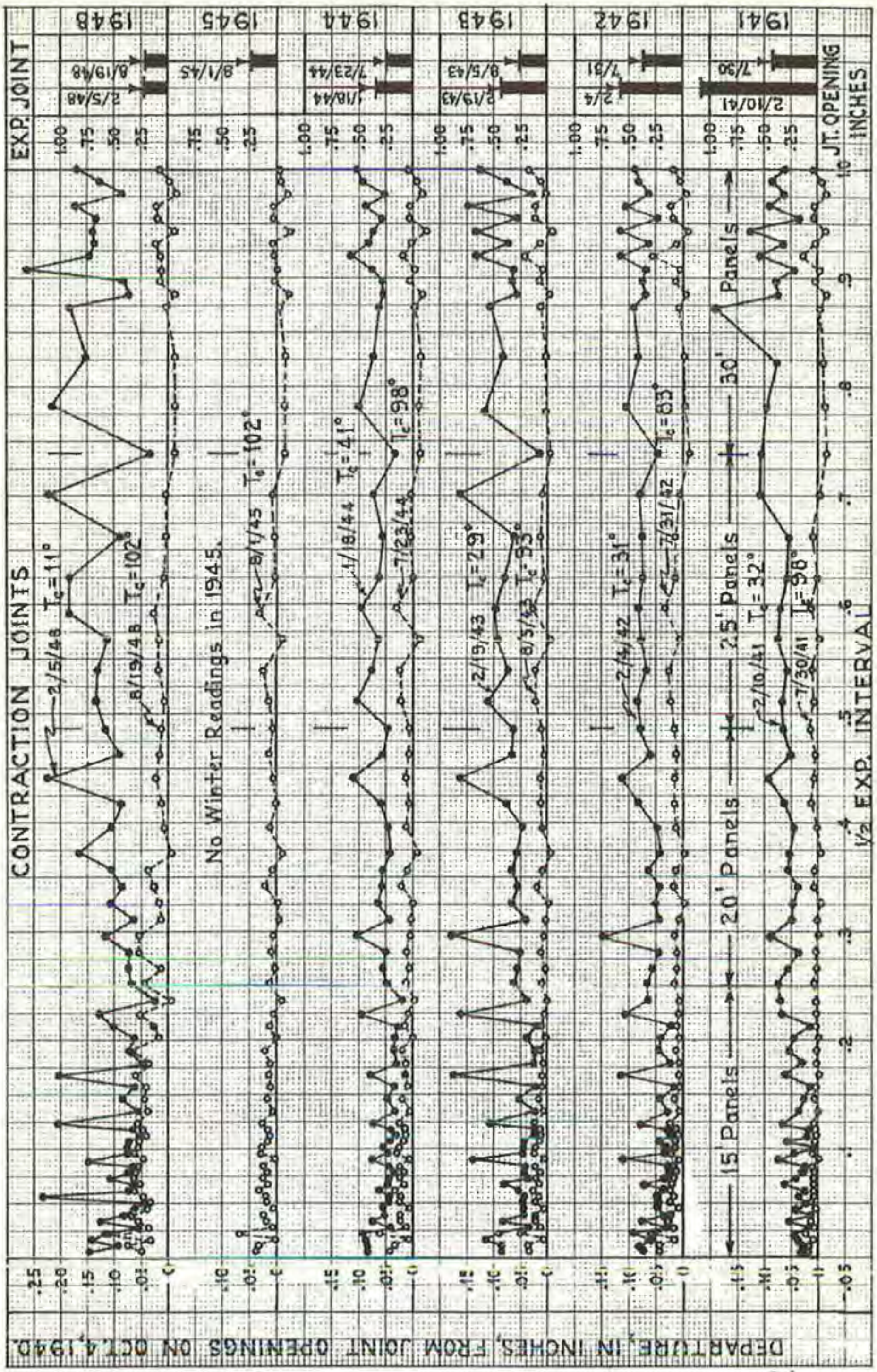


Figure 24. Annual and permanent changes in joint openings. West ½ of div. 9 - Sta. 400 + 51.6 to Sta. 426 + 82.1.

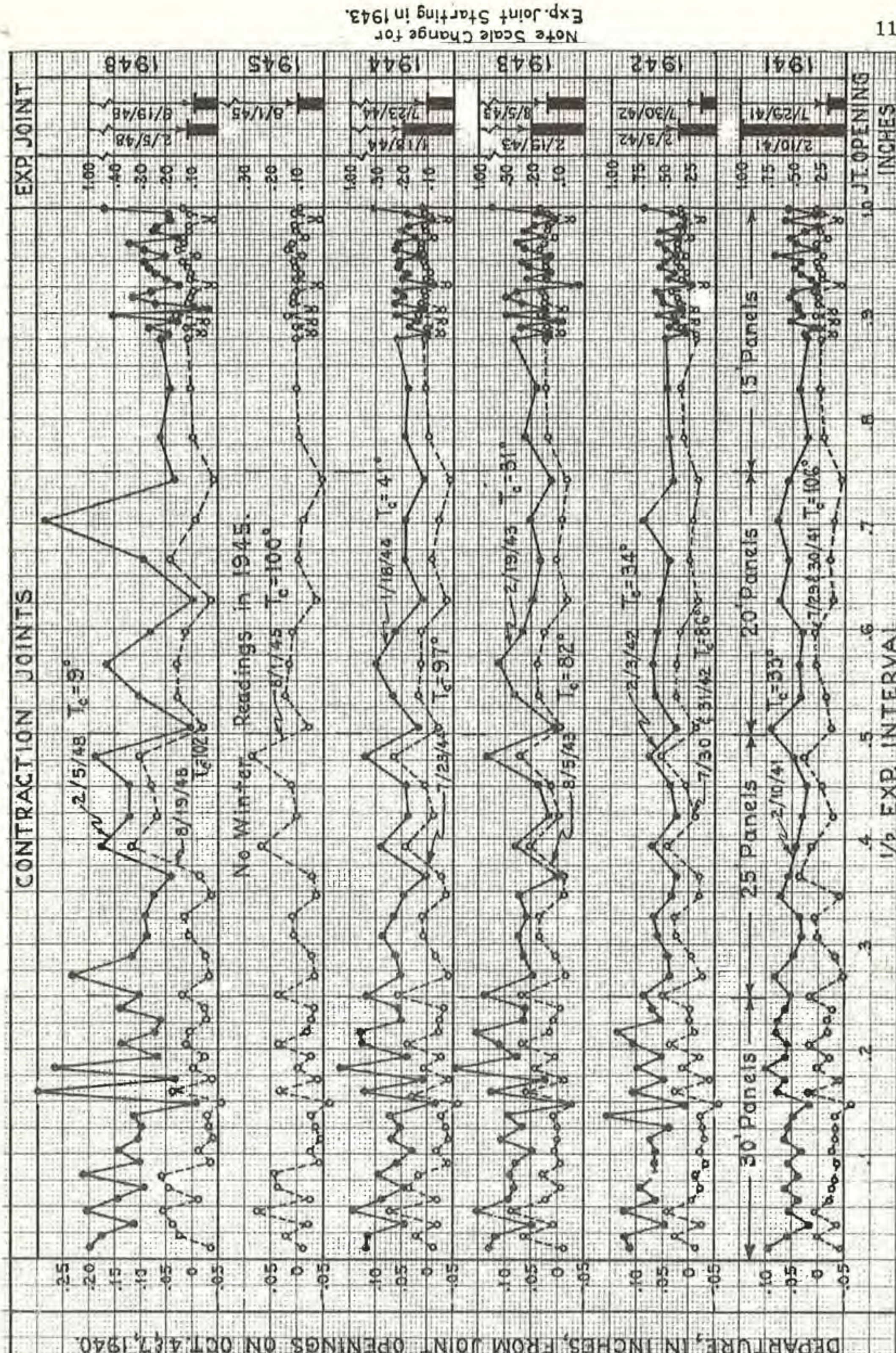


Figure 25. Annual and permanent changes in joint openings. East ½ of div. 9 - Sta. 426 + 82.1 to Sta. 453 + 11.7.

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

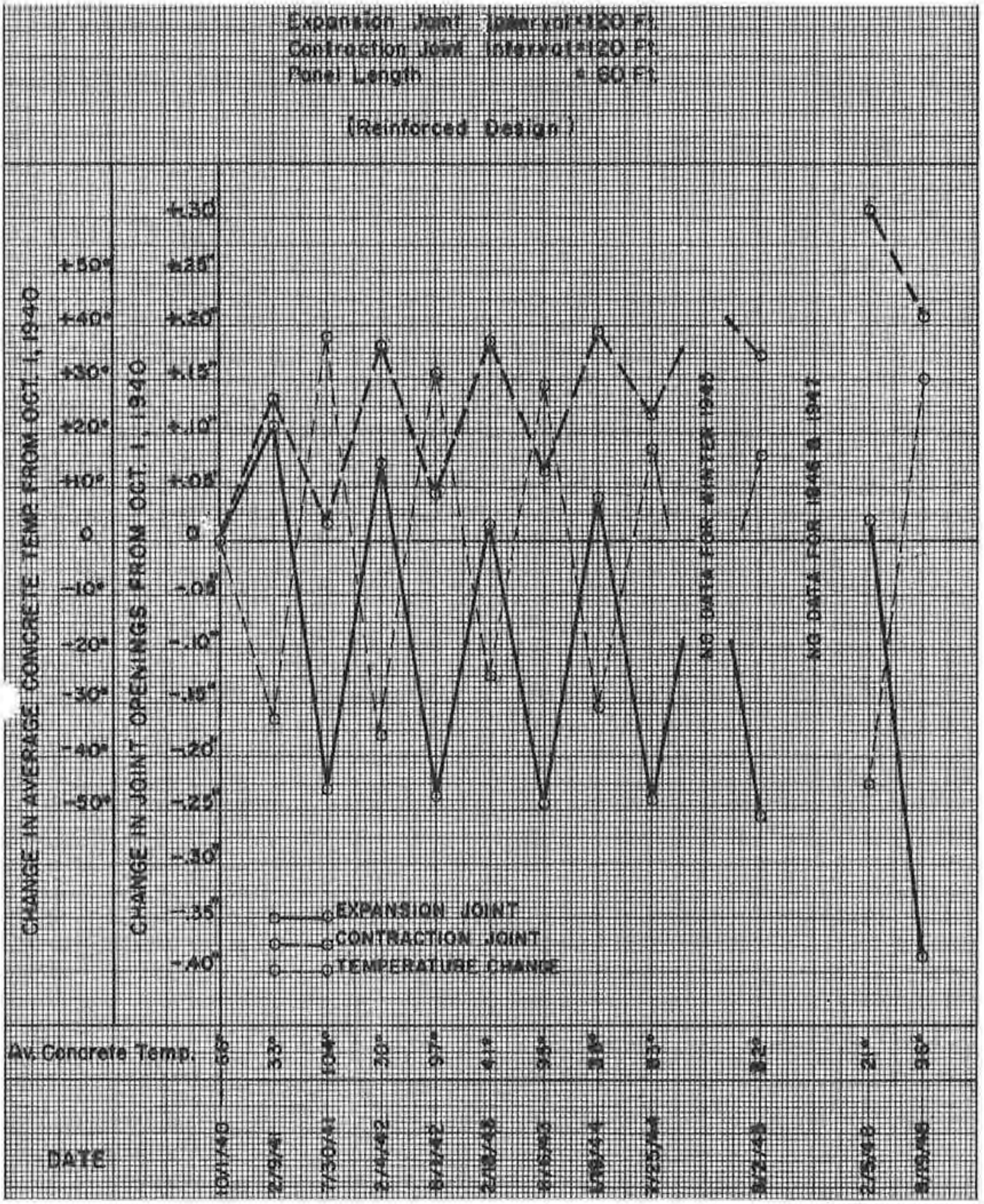


Figure 26. Annual and permanent changes in joint openings.

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

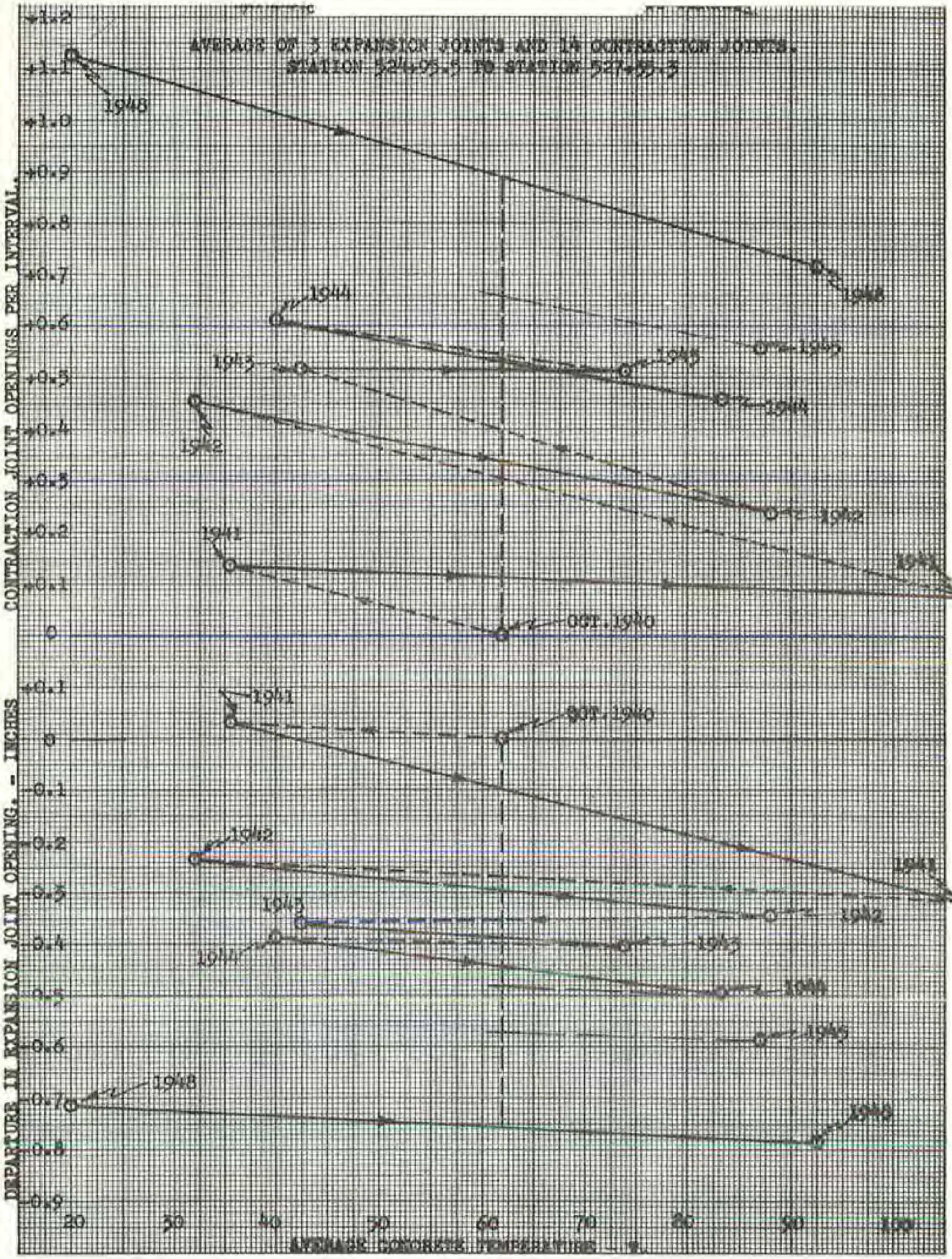


Figure 27. Annual and permanent changes in joint openings. Expansion interval of 120 feet - 15 foot panels.

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

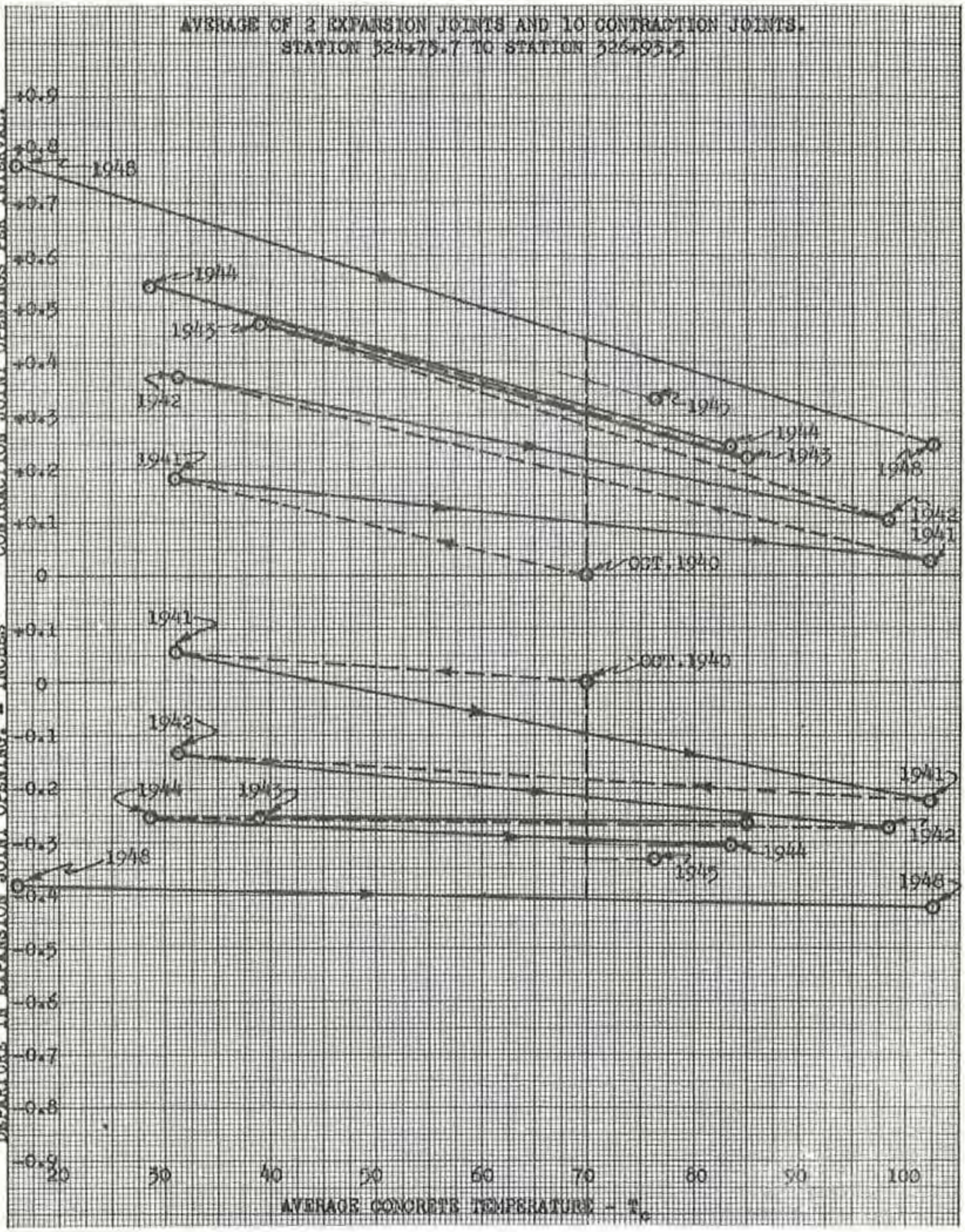


Figure 28. Annual and permanent changes in joint openings. Expansion interval of 120 feet - 20 foot panels.

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

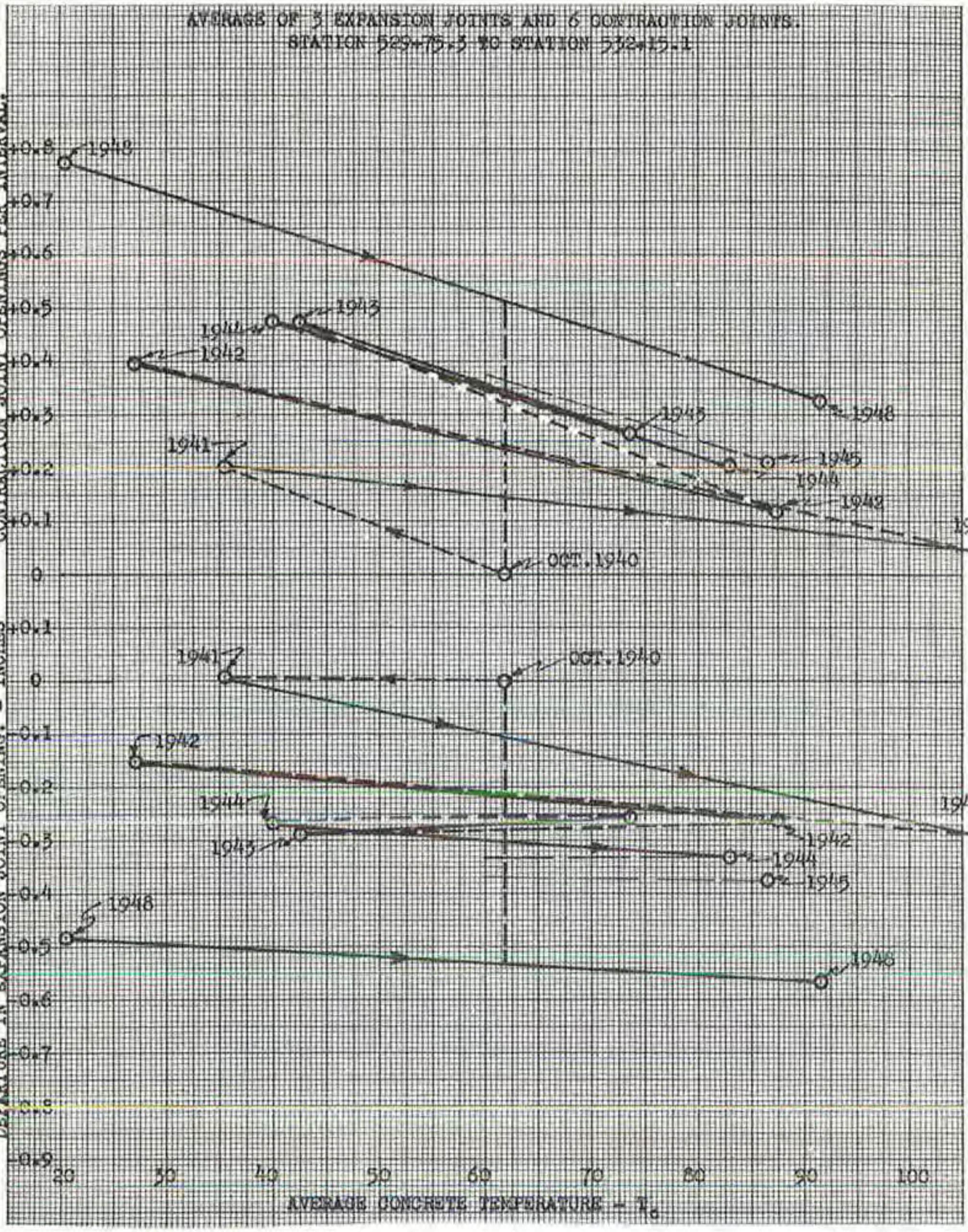


Figure 29. Annual and permanent changes in joint openings. Expansion intervals of 120 feet - 30 foot panels.

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^{0}F. 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.

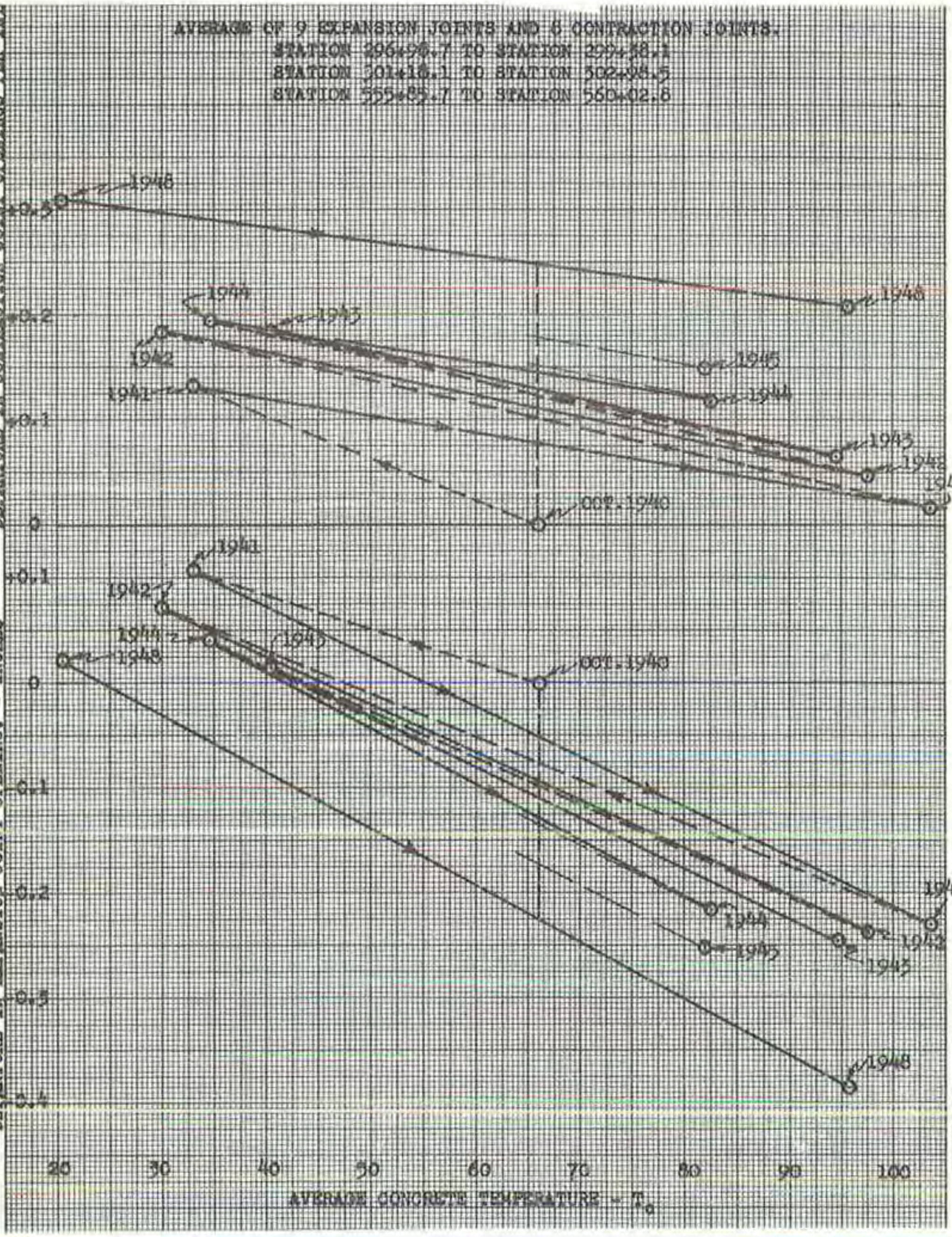


Figure 30. Annual and permanent changes in joint openings. Expansion interval of 120 feet - 60 foot reinforced panels.

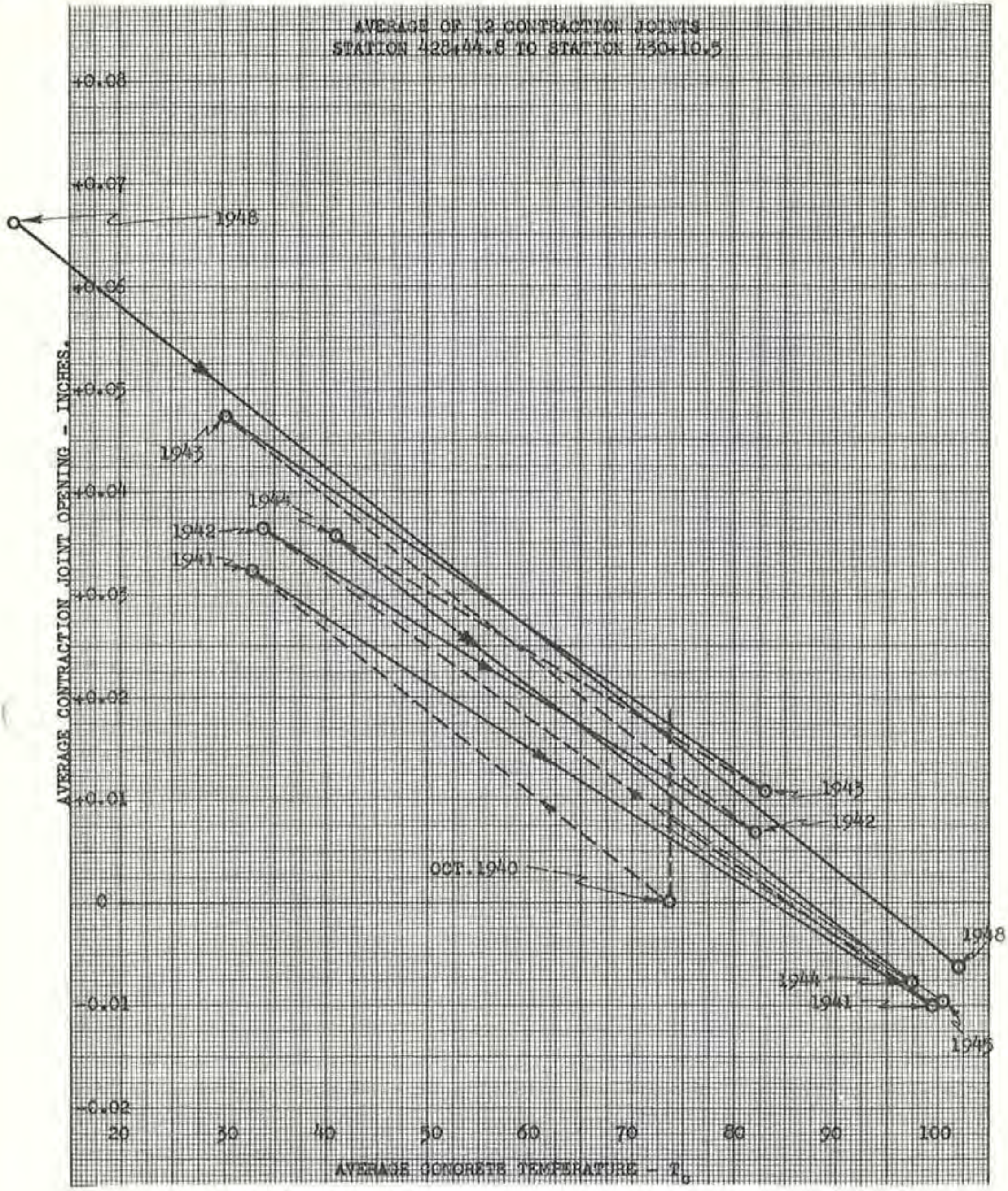


Figure 31. Annual and permanent changes in joint openings. Contraction joint interval of 15 feet near center of division 9.

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-

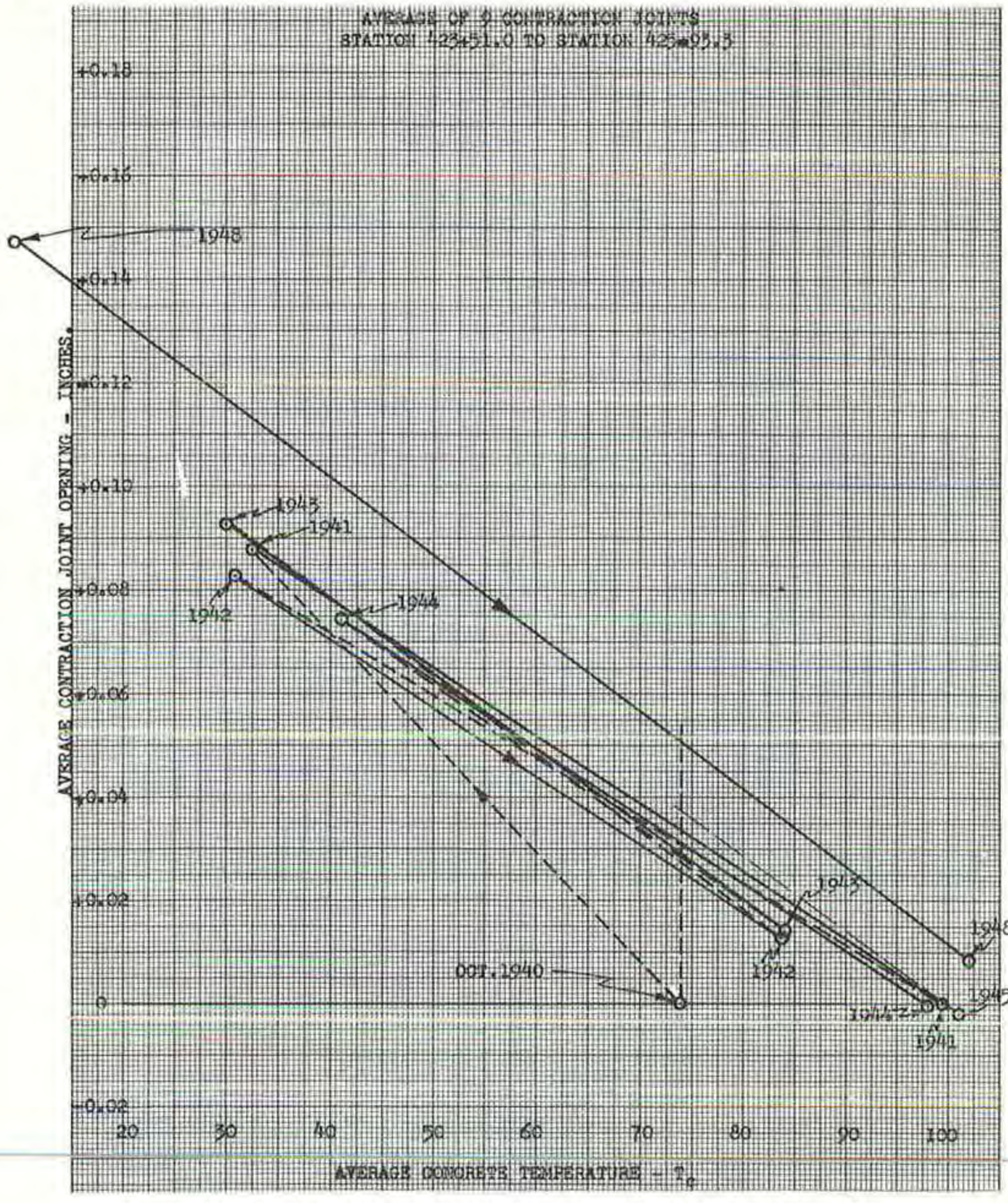
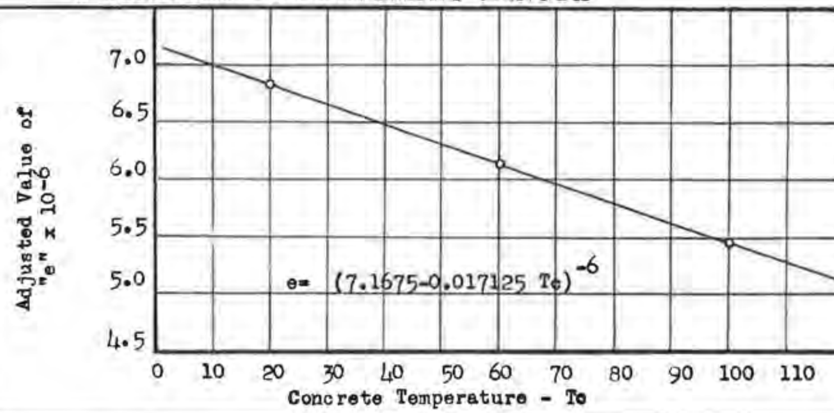


Figure 32. Annual and permanent changes in joint openings. Contraction joint interval of 30 feet near center of division 9.

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 $54^{0}+41.1^{0} = 95.1^{0}$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^{0}F. above hardening temperature or 107.8^{0}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	*	*	*	*	-	.044	*	*
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-

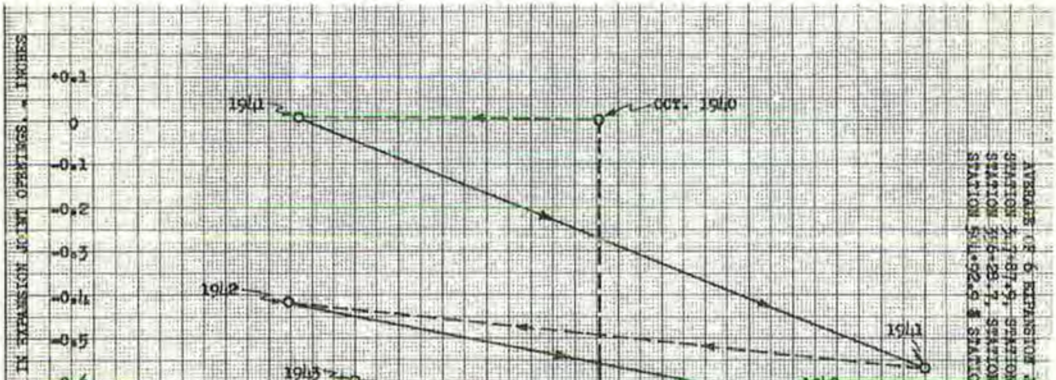


Figure 33. Annual and permanent changes in joint openings. Expansion joint interval of 420 feet.

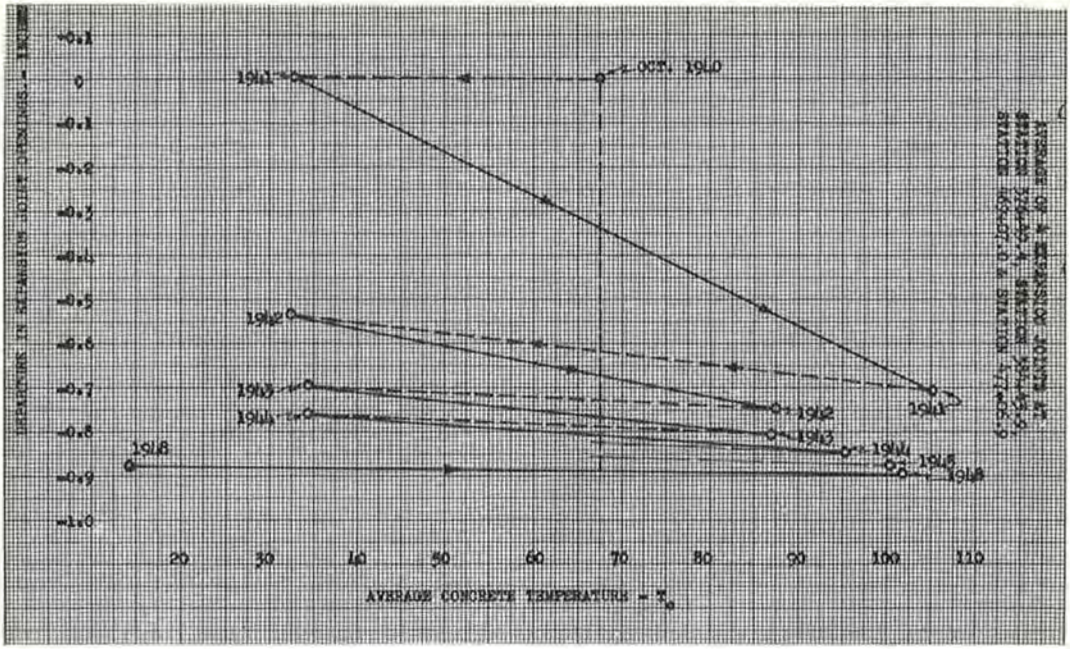


Figure 34. Annual and permanent changes in joint openings. Expansion joint interval of 800 feet.

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

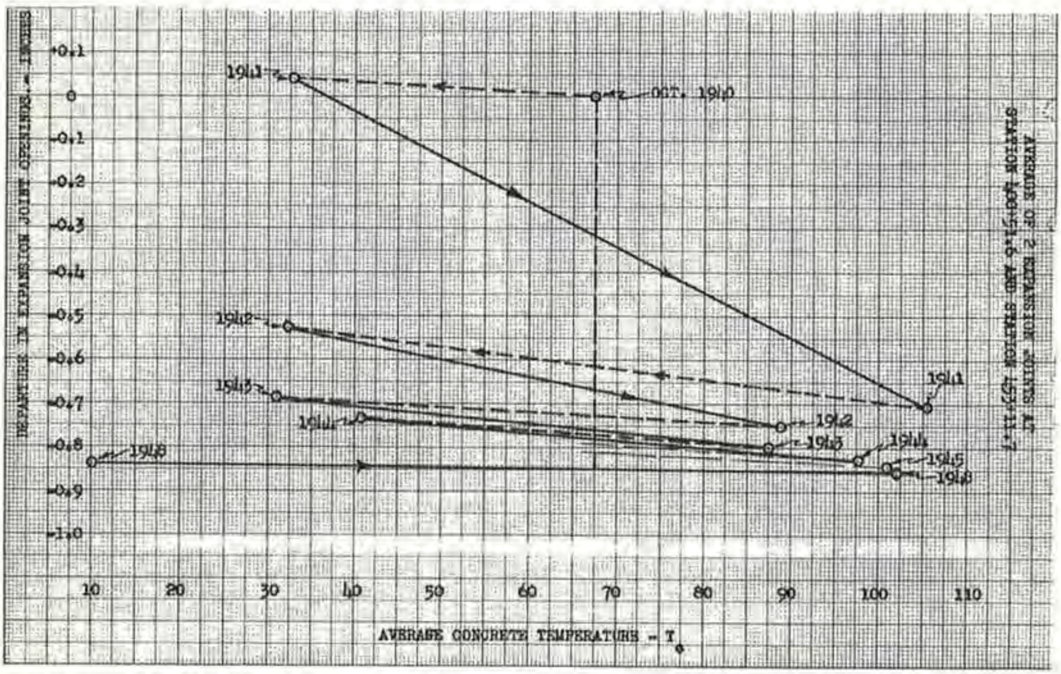


Figure 35. Annual and permanent changes in joint openings. Expansion joints at ends of division 9.

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.

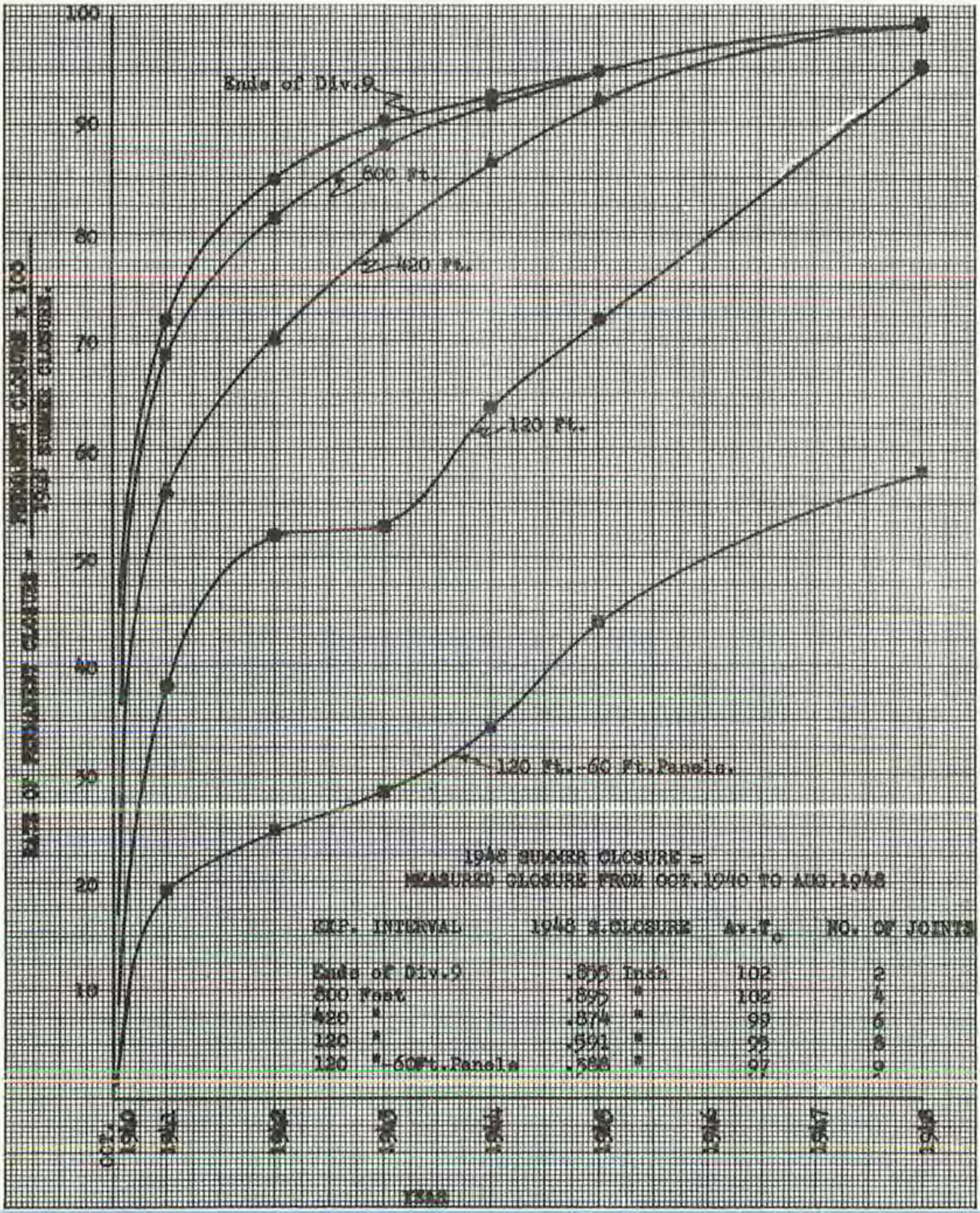


Figure 36. Rate of permanent closure of expansion joints for various expansion intervals.

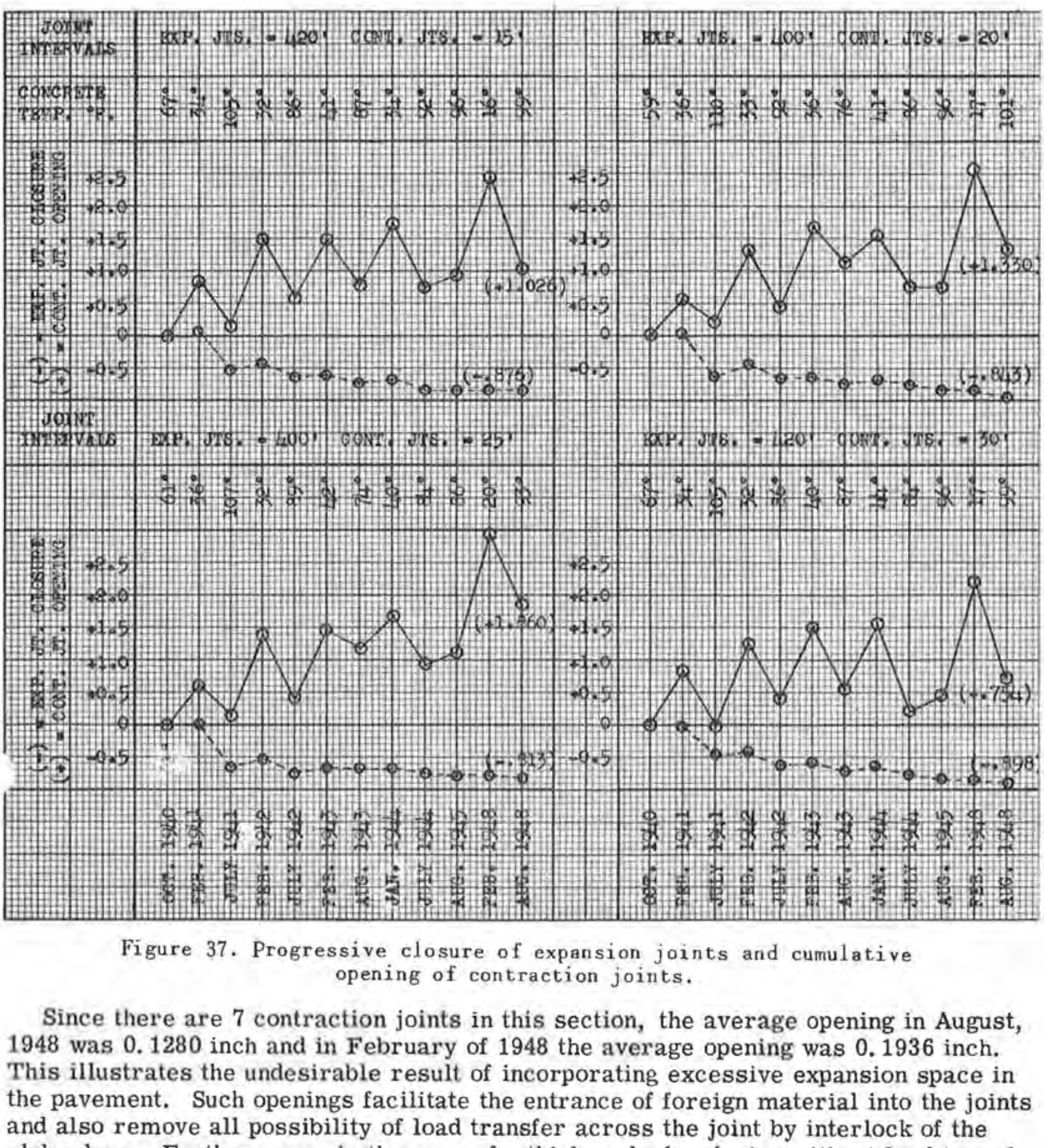


Figure 37. Progressive closure of expansion joints and cumulative opening of contraction joints.

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

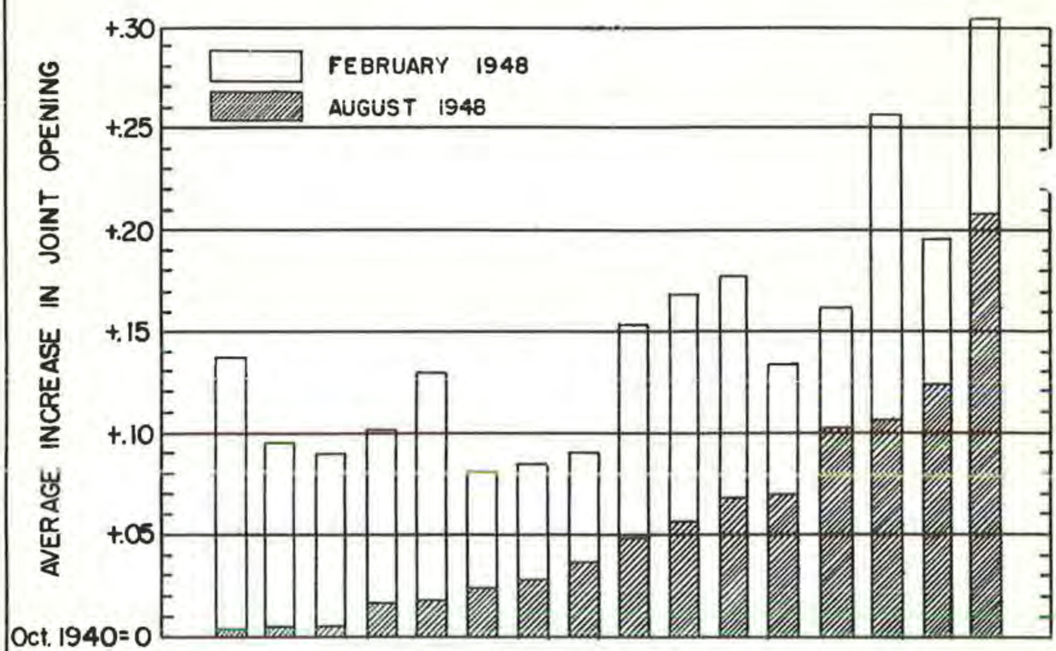
Possibly due to subgrade condition.

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

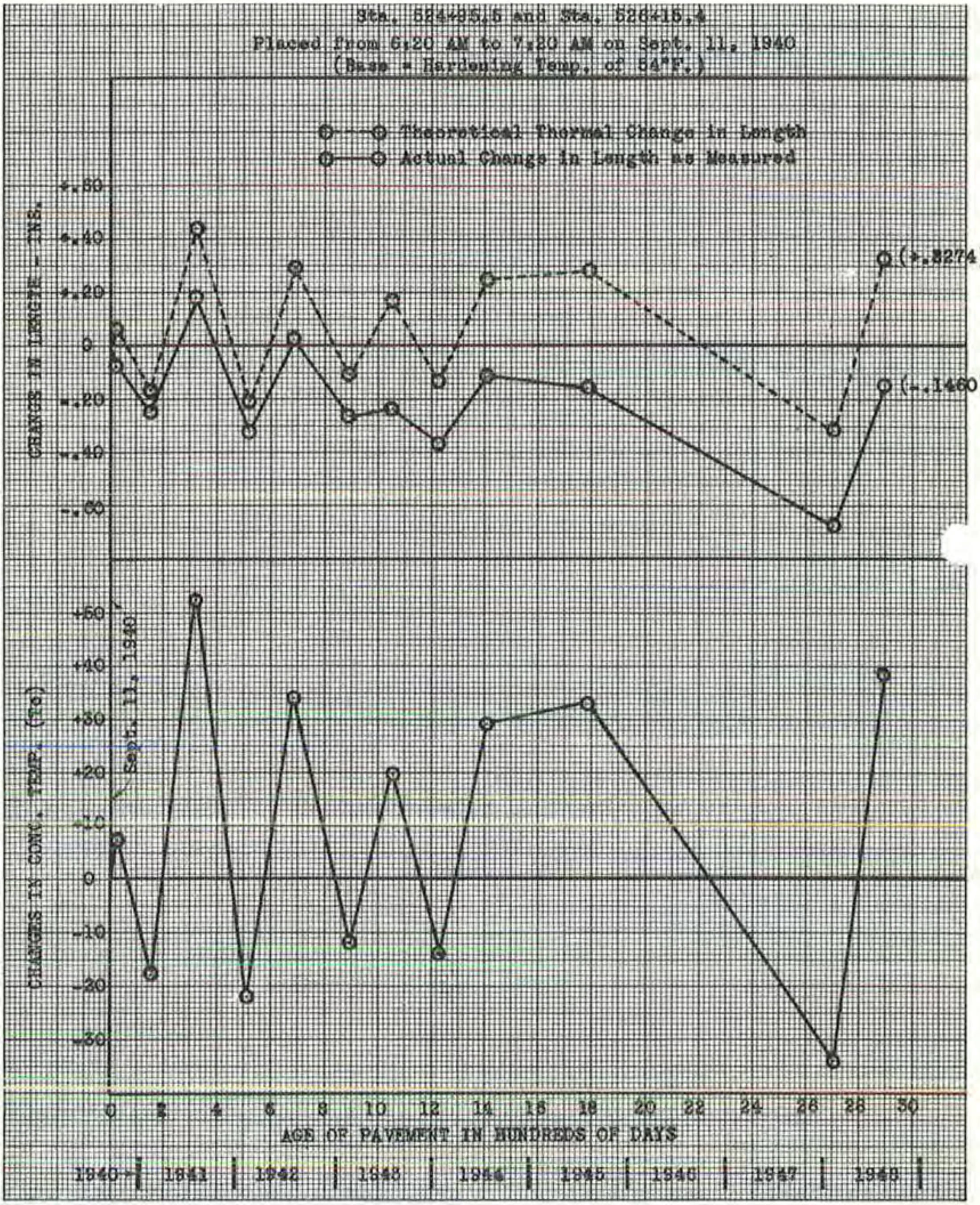


Figure 39. Annual changes in concrete temperature and length of 120 ft. of pavement between expansion joints.

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 $\frac{1}{4}$ 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 $\frac{1}{4}$ 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$\pm$°F. to 95$\pm$°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 $\frac{1}{4}$ 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.

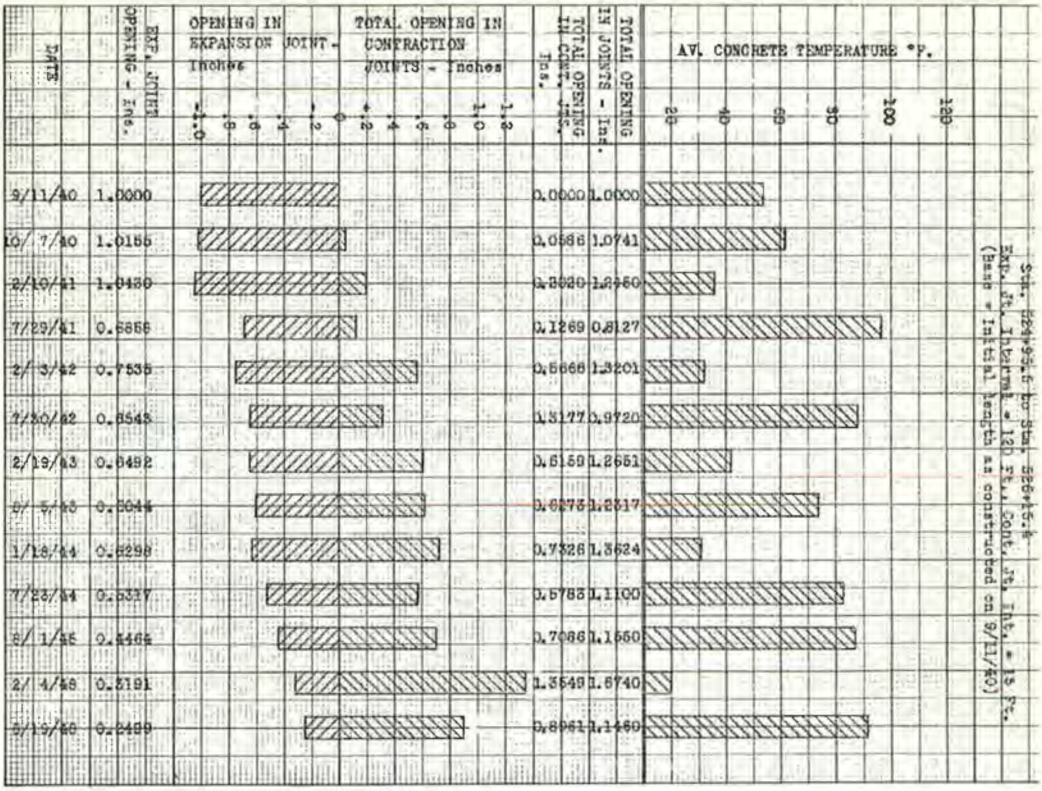


Figure 40. Progressive Closure of expansion joints and cumulative opening of contraction joints.

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%.

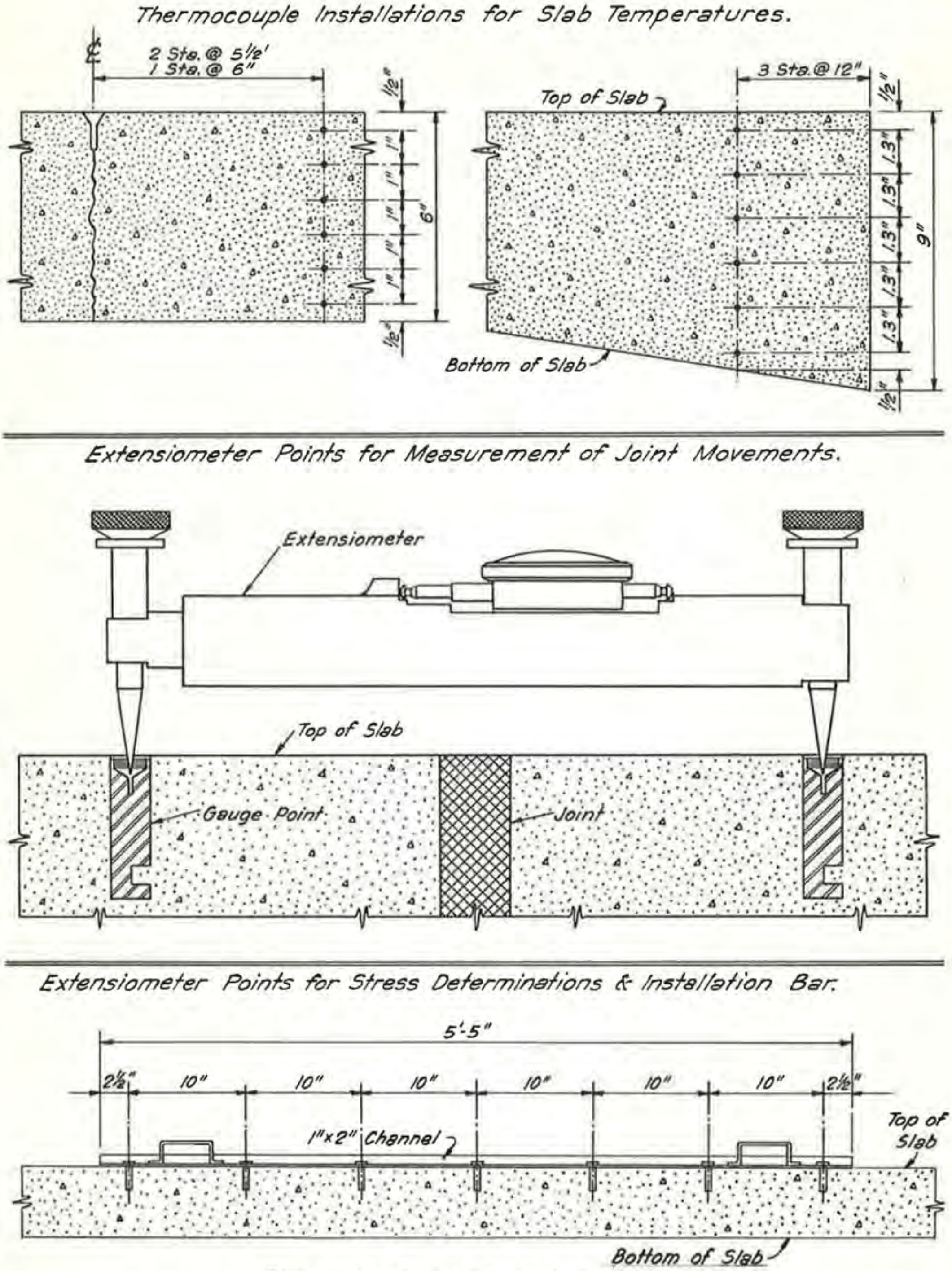


Figure 41. Instrumentation details.

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 = $4,300,000 \times K_1 \times 10^{-5}$.

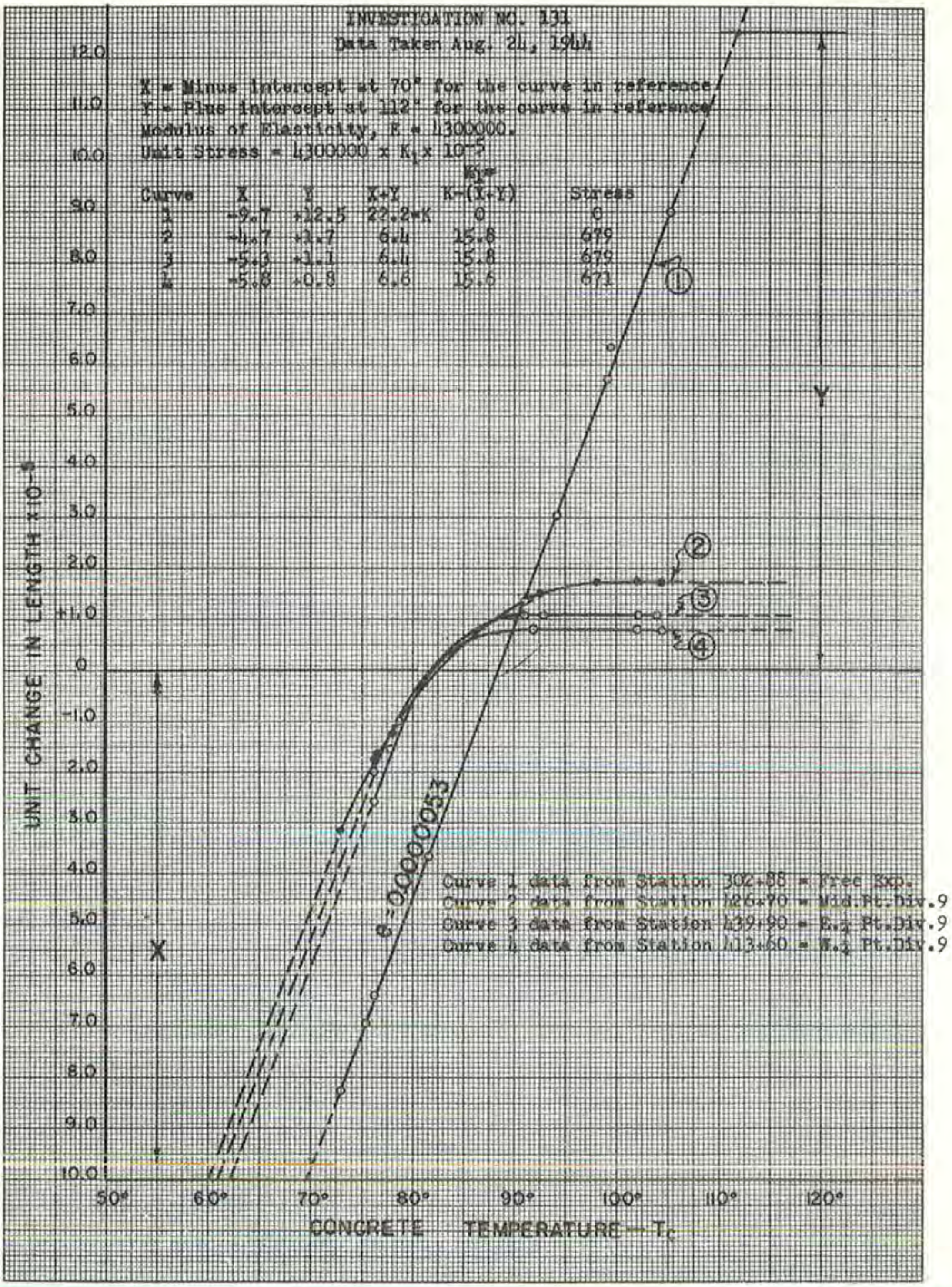


Figure 42. Unit stress determinations.

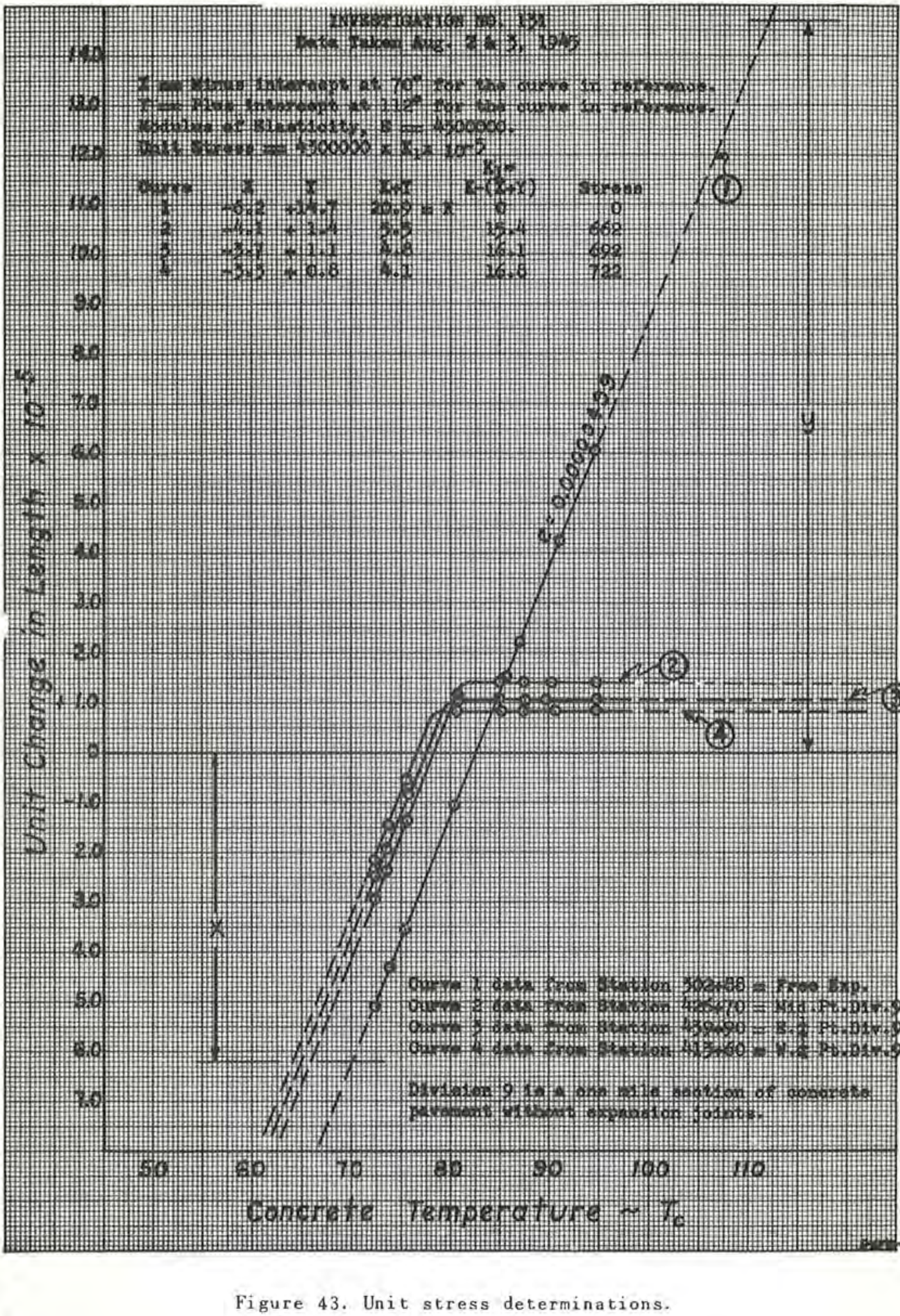


Figure 43. Unit stress determinations.

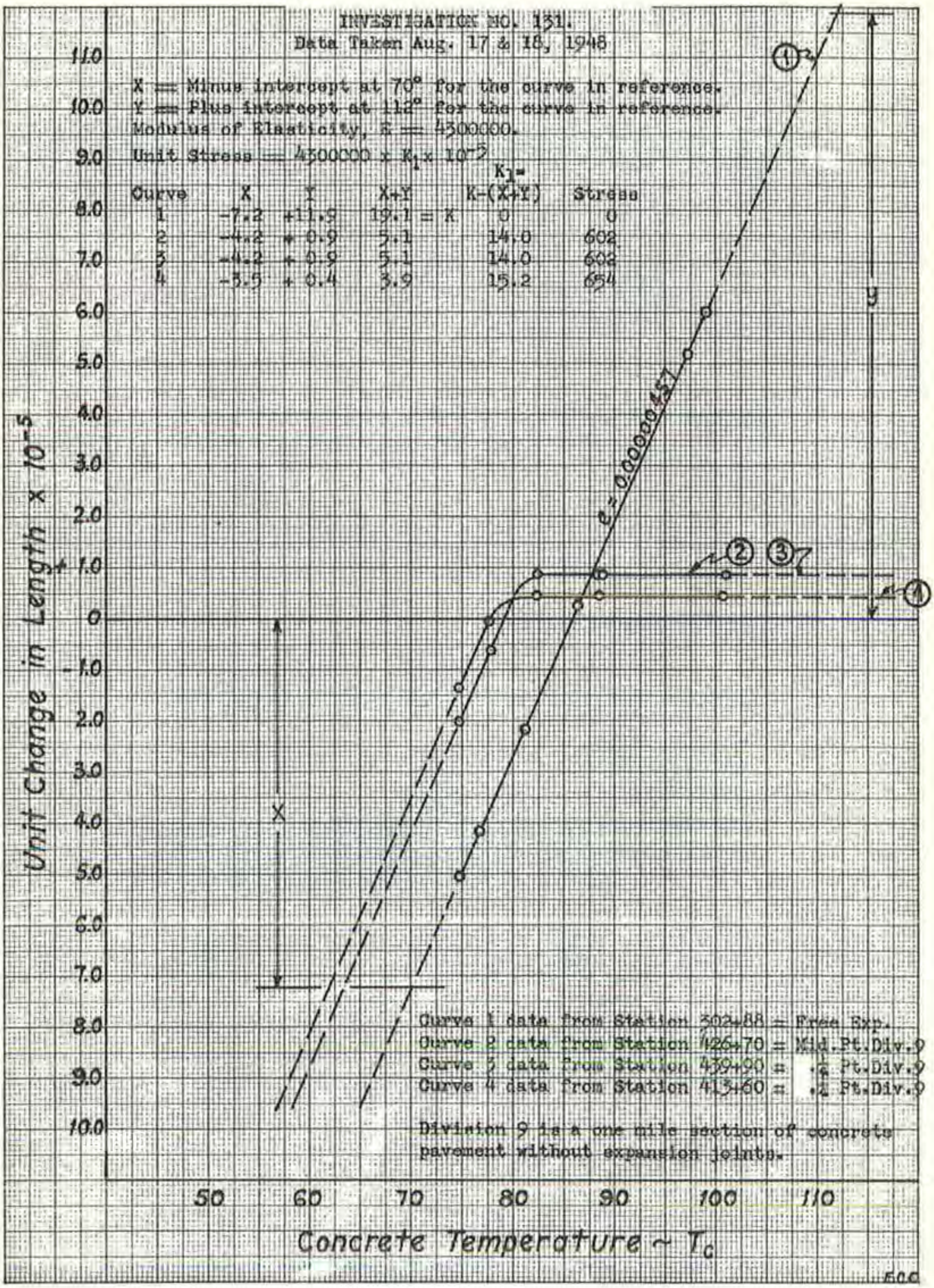


Figure 44. Unit stress determinations.

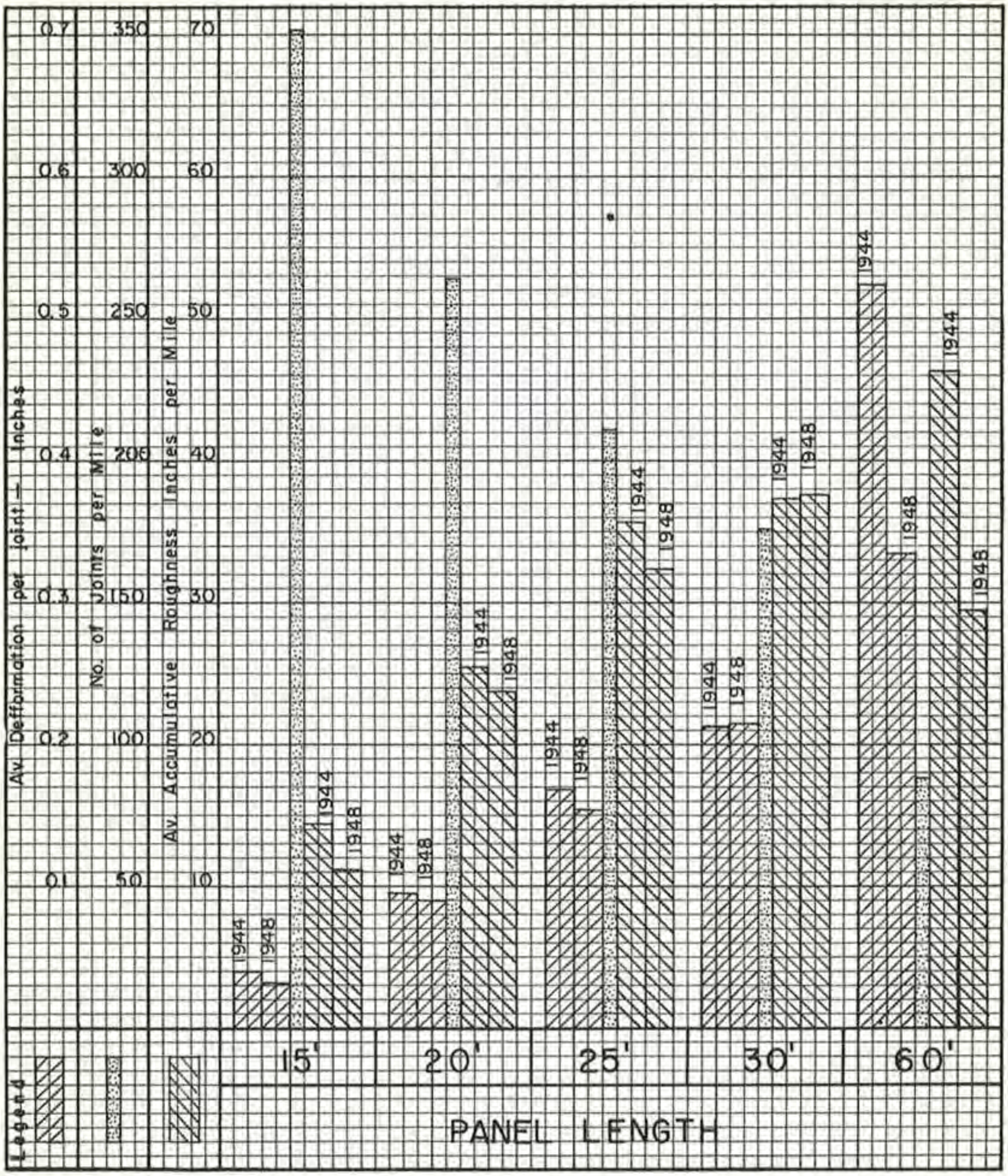


Figure 45. Effect of panel length on roughness.

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 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.

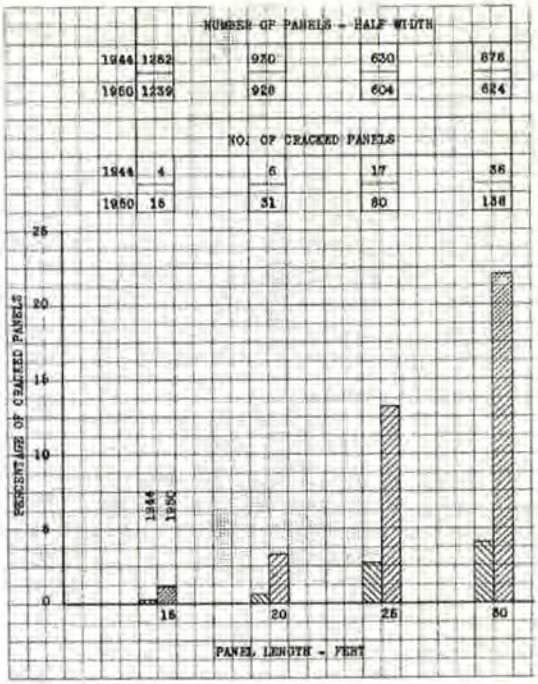


Figure 46. Relationship of transverse cracking to panel length.

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 mid-point 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	Per-cent 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 $\frac{1}{4}$ 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

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