INVESTIGATIONAL CONCRETE PAVEMENT IN MINNESOTA

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The general layout and the special features of design of this experimental pavement were described at the Twentieth Annual Meeting of the Highway Research Board (1940).[1]

This report covers pertinent construction details together with data on observations and measurements taken up to, and including those of July, 1944.

CONCRETE

The aggregates used on this project were washed sand and gravel. These materials were shipped to the project by rail and were proportioned by weight from a track-side proportioning plant The cement was a standard Type I cement.

The mix proportions, by absolute volumes, were maintained nearly constant, the range being from approximately 1:2 8:6 4 to 1:2.9:6.1. The water-cement ratio ranged from approximately 5.8 to 6 1 gal per sack; and the consistency of the concrete was maintained within a range of ½ to 1½ in. of slump.

The concrete was consolidated by means of a full-width tubular vibrator of the internal type, which operated at a frequency of 4500–5000 impulses per minute.

Tables 1 and 2 show the properties of the concrete and the results of various strength tests.

TRAFFIC

The amount and character of the traffic on this project subsequent to pavement construction as well as the increase in the heavy semi-trailer type of vehicle during the past seven years is shown in Table 3

CLIMATOLOGICAL DATA

Average maximum and minimum temperatures, based on the 10 yr. period prior to construction (1930–1940) were shown in the 1940 report. Monthly precipitation data, based on the average of the period from 1891 to 1940, were also shown in the 1940 report.

Figure 1 shows the monthly maximum, minimum, and mean temperatures from August, 1940 to July, 1944 inclusive. These data cover the construction period from August 6 to September 20, 1940, as well as the period subsequent to construction during which observations and measurements have been made Figure 2 shows, for the same period, the monthly precipitation data

DAILY CHANGES IN JOINT OPENINGS

Daily changes in width of opening have been measured at 70 joints on this project. Of these, 18 were expansion joints and 52 were contraction joints These measurements have been made at 3-hr. intervals throughout a 26-hr. period both winter and summer during each year since the pavement was built

These joints were selected in such a way as to provide data on joint openings over different expansion joint intervals ranging from 120 ft. to approximately one mile Various contraction joint intervals, or panel lengths, were also involved in this series of joint measurements, including intervals of 15, 20, 25, 30, and 60 ft.

During the past 4 years a large number of these measurements have been made; and these are illustrated by the average values of daily change in openings for one day (July 24, 1944) which are included in this report. These data are shown in Table 4

Table 4 contains only the average total daily change in openings and does not show the relationship between width of opening and the changing daily temperature or the relative position of the joint This is illustrated by the two examples shown in Figures 3 and 4 Figure 3 shows the change in opening width as well as the temperature change at different times during the 24-hr. period for a section 120 ft between expansion joints and made up of two 60-ft panels Figure 4 shows similar data for a section 420 ft. between expansion joints and made up of a series of 15-ft panels Both of these figures show the tight closure of the intermediate

[1] "F. C Lang, Investigational Concrete Pavement in Minnesota," *Proceedings*, Highway Research Board, Vol 20, p 348 (1920)

TABLE 1
CONCRETE STRENGTH TESTS

Flex. Tests Made in Field (**)

STA - STA.	Pit No. 1, 7 Day Tests: No. Tests	Pit No. 1, 7 Day: Av Unit Str	Pit No. 1, 7 Day: Std. Dev. (*)	Pit No. 1, 7 Day: Coef. of Var (*)	Pit No. 1, 14 Day Tests: No. Tests	Pit No. 1, 14 Day: Av Unit Str	Pit No. 1, 14 Day: Std. Dev (*)	Pit No. 1, 14 Day: Coef of Var. (*)	Pit No. 2, 7 Day Tests: No. Tests	Pit No. 2, 7 Day: Av Unit Str	Pit No. 2, 7 Day: Std. Dev (*)	Pit No. 2, 7 Day: Coef of Var (*)	Pit No. 2, 14 Day Tests: No. Tests	Pit No. 2, 14 Day: Av Unit Str.	Pit No. 2, 14 Day: Std Dev (*)	Pit No. 2, 14 Day: 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	Age of Cores (***): When Tested	Center Cores: No. Cores	Center Cores: Height (Ins) Theor	Center Cores: Height (Ins) Act	Center Cores: Std Dev (*)	Center Cores: Coef of Var (*)	Side Cores: No. Cores	Side Cores: Height (Ins) Theor	Side Cores: Height (Ins) Act	Side Cores: Std. Dev (*)	Side Cores: Coef of Var (*)	Compressive Strength (Corrected for H/d): No. Cores	Av Unit Str	Std Dev (*)	Coef of Var (*)
222+68 264+30 3 293+38 5 376+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%	13	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+00 222+68 376+64.4 563+63 592+72 6 634+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	8.82%

Note: 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.

contraction joints during the warmest part of the 24-hr. period.

The width of any individual contraction joint is continually changing. It is affected not only by the daily and seasonal changes in temperature and moisture, but also by the length of adjoining panels, the length of the expansion joint interval, and the position of the joint on this interval, with reference to the nearest expansion joint. It is also affected by the age of the pavement, the number of annual cycles of expansion and contraction, and the degree of closure of adjacent expansion joints, prior to the time of taking the measurement.

In general, the daily change in the widths of contraction joints decreased as the expansion joint interval increased and the

TABLE 2
MISCELLANEOUS CONCRETE PROPERTIES AND 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+644 - 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	4249	11	5142	11	4738	10	5564	10	4219
207+00 - 222+68 376+644 - 634+64	Pit No. 2	19	4829	20	5252	20	5166	22	5341	17	5835

ontraction joint interval decreased. Short anels and wide spacing or omission of ex- ansion joints therefore would appear to be eneficial in preventing or reducing leakage of urface water through the joints and also in

TABLE 3

Vehicle Type	Average Maximum Axle Load[a]	Number per 24 hours		
		1937[b]	1941[b]	1944 (May)
assenger Automobiles				395
ight Trucks	7,673			96
edium Trucks	11,249			25
eavy Trucks	19,200			0
ractor-Truck Semi-Trailers	14,762	3	13	48
usses				2
Totals				602

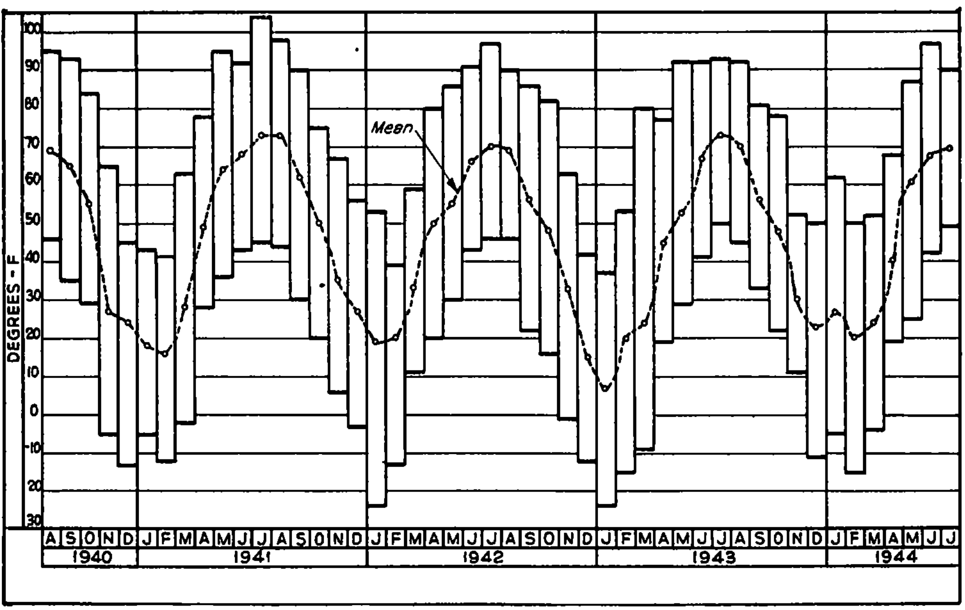
[a] Based on traffic study made at 10 loadometer stations uring 1943, except axle load of heavy trucks, which is based n 1942 study

[b] Annual Averages

for the purpose of determining the annual, or seasonal, and the permanent changes in joint openings. Included in this group were 99 expansion and 615 contraction joints A complete set of initial measurements was made early in October of 1940, which was about two weeks after completion of the pavement Since then, measurements have been made as follows:

1941—February, May, July, November
1942—February, May, July and August
1943—February and August
1944—January and July

The data which have been accumulated during this period are too extensive to be shown with any degree of detail in this short report; therefore, only four examples, in graphical form, are shown in Figures 5, 6, 7, and 8.



igure 1. Air Temperature Data—Monthly Minimums, Maximums & Means, August 1940–July 1944 Incl., Worthington, Minnesota

roviding a maximum degree of load transfer cross the joints.

ANNUAL AND PERMANENT CHANGES IN JOINT OPENING

Measurements have been made during the ast four years on a group of selected joints

Figure 5 shows the change in opening of individual joints on a 120-ft. section; i e, 120 ft between expansion joints, on which the contraction joint interval was 20 ft The horizontal scale under "Contraction Joints" in this figure is of the folded type and represents, to decimal scale, one-half the length of the section. The plotted points are the averages of the corresponding joints on both halves of the section Each block of the plotting represents data for a given year as indicated along the right hand margin individual joints for each of the four years, and also it shows the progression in the permanent changes to date.

Figure 6 shows in the same form of plotting the data for a 400-ft. section having 25-ft.

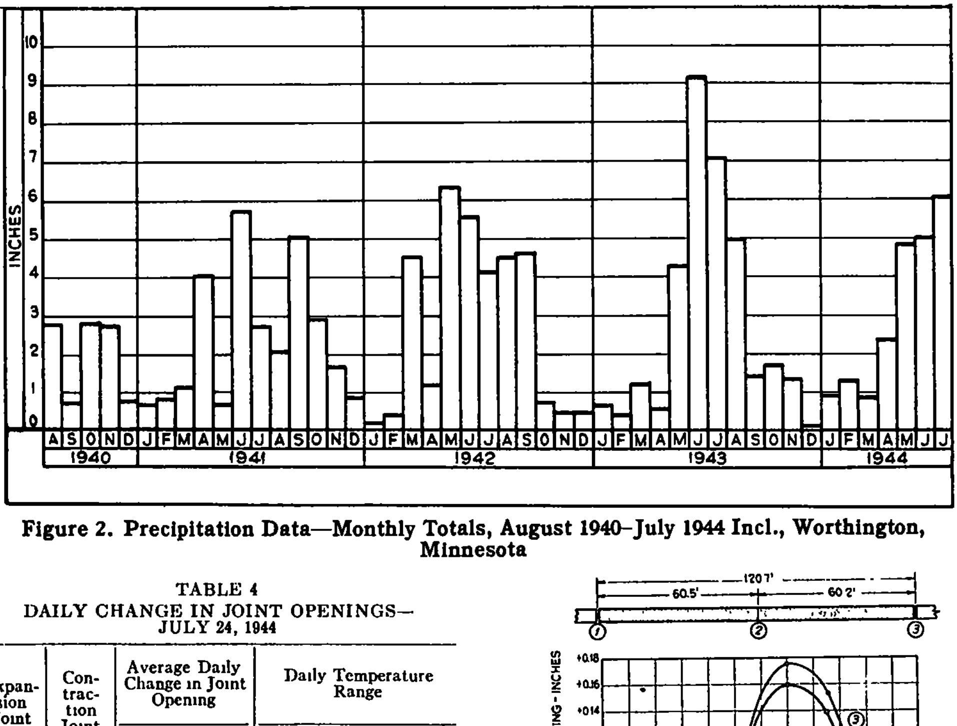


Figure 2. Precipitation Data—Monthly Totals, August 1940–July 1944 Incl., Worthington, Minnesota

TABLE 4
DAILY CHANGE IN JOINT OPENINGS—JULY 24, 1944

Expansion Joint Interval	Contraction Joint Interval	Average Daily Change in Joint Opening		Daily Temperature Range		
		Contraction	Expansion	From	To	Range
ft	*ft*	*in*	*in*	*deg F*	*deg F*	*deg F*
120	30	067	068	73	106	33
120	60[a]	075	171	73	106	33
125	25	060	023	73	106	33
420	15	034	065	73	106	33
420	30	.058	055	73	106	33
795	15	.032	016	73	106	33
810	30	034	020	73	106	33
5,260	15	030	025	73	106	33
5,260	30	048	025	73	106	33

[a] Reinforced Panels

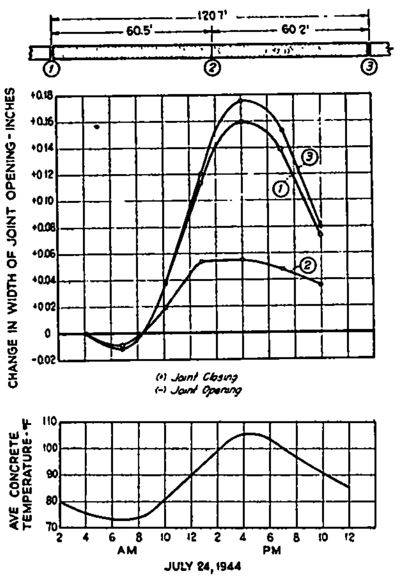


Figure 3. Daily Change in Joint Width—Station 301 + 77.8 to Station 302 + 98.5.

The zero end of the horizontal scale represents the expansion joint end of the half-section and the scale value of 1 0 represents the midpoint of the section On the right side of the plotting the measured changes in opening of the associated expansion joints are shown.

This form of plotting the data shows the seasonal changes that have taken place at

spacing of contraction joints. Figure 7 shows the data for an 810-ft. section with 30-ft. contraction spacing

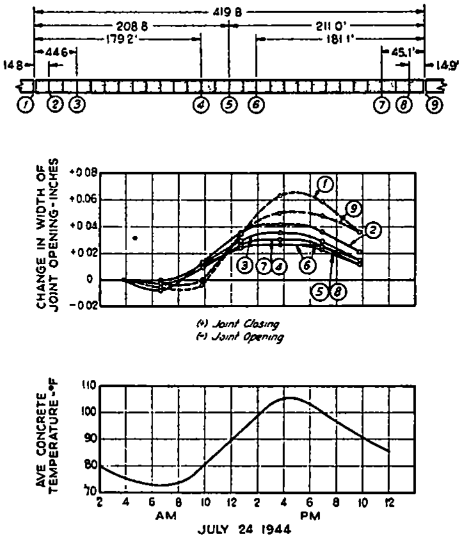


Figure 4. Daily Change in Joint Width—Station 352 + 08.9 to Station 356 + 28.7.

Because only one contraction joint is involved, the data for a typical 120-ft. section with 60-ft. reinforced panels, are shown in a different manner in Figure 8. This form of plotting is particularly adaptable to showing the seasonal and permanent changes in opening of individual joints, and it permits a graphical determination to be made of the permanent change at any common temperature, i e, it provides an easy way to correct for variations in temperature, as indicated by the heavy vertical dashed lines on the 66-deg ordinate

The plottings in Figures 5, 6, and 7 show the general tendency for the end contraction joints next to expansion joints to develop openings which are relatively much wider than the openings in contraction joints more distant from the expansion joints. This is due, of course, to the fact that the end panels are displaced on the subgrade by the accumulated expansion of the interior panels The

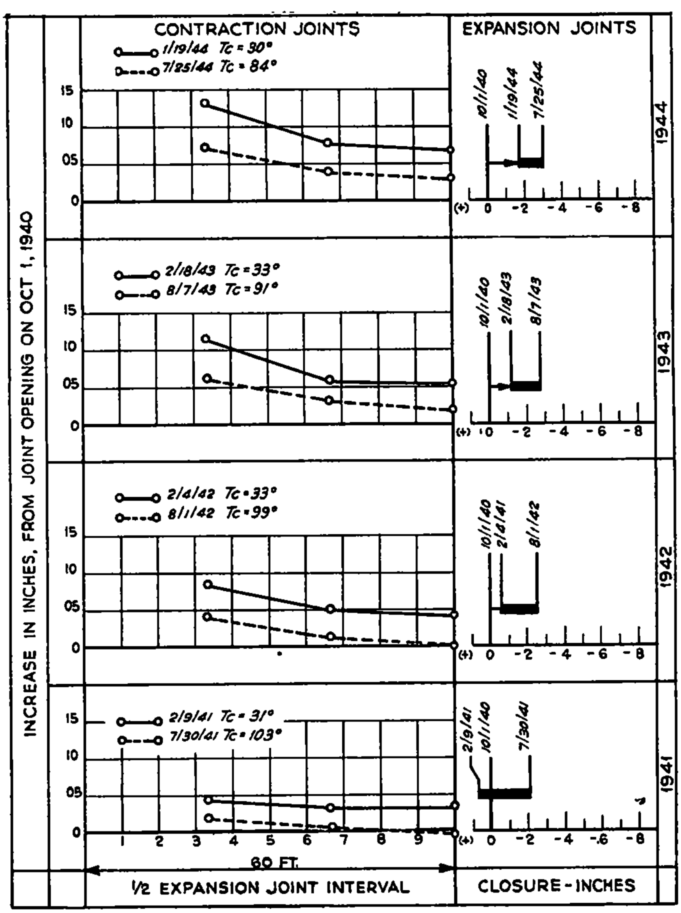


Figure 5. Annual & Permanent Changes in Joint Openings. Expansion Joint Interval = 120 ft. Contraction Joint Interval = 20 ft. (Base = Oct. 1, 1940)

plottings also indicate the somewhat erratic behavior of individual contraction joints over the whole section with respect to the width of their openings. Some joints show openings considerably wider than others at a given time This is probably caused principally by variation in the depth of the weakened-plane impression of the joints at the time the possible, too, that imperfectly placed dowels and variations in subgrade roughness may have been contributing factors

Plottings for all the combinations of expansion and contraction joint spacing cannot be shown in this report; but the general effect of these factors is indicated on Figure 9, which shows the accumulated average change

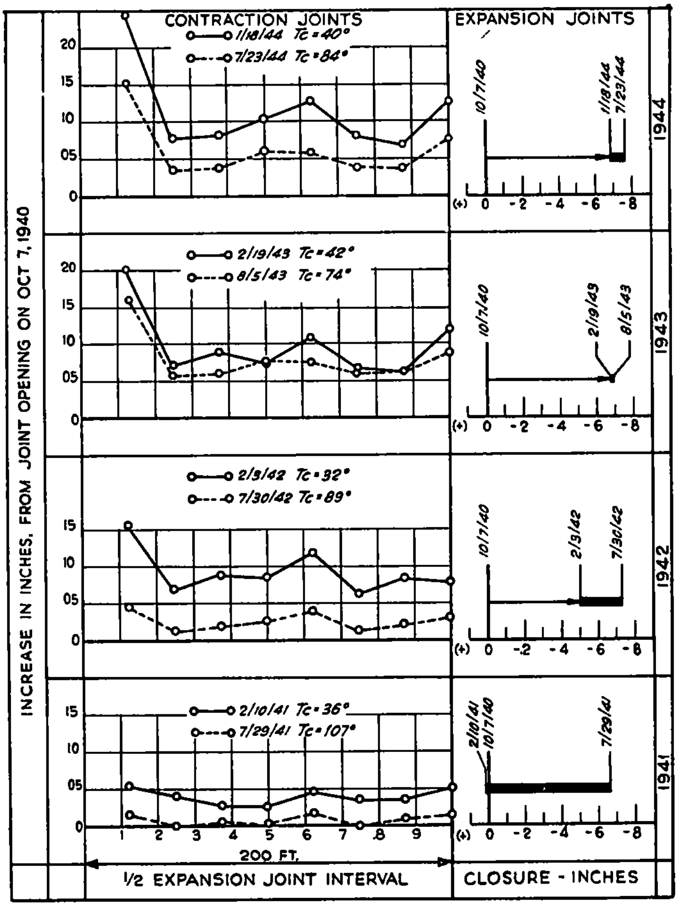


Figure 6. Annual & Permanent Changes in Joint Openings. Expansion Joint Interval = 400 ft. Contraction Joint Interval = 25 ft. (Base = Oct. 7, 1940)

concrete hardened Measurements made of the impression depth on a considerable number of joints showed depths varying from $\frac{3}{4}$ in. to the planned depth of $2\frac{1}{4}$ in The variation in depth was caused by too early removal of metal inserts and to rainfall, both of which resulted in sloughing of the concrete and partial closure of the impression It is in contraction joint openings for all the combinations of expansion and contraction joint spacings included on this project This shows the advantage of both short panels and wide spacing, or omission, of expansion joints in providing small openings in contraction joints The average opening of contraction joints spaced 15 ft apart on Division

9 (1 mile without expansion joints) was less than 0 05 in. even in January of 1944, which was 4 yr. after construction In July of 1944, the openings were only slightly greater than they were in October, 1940, just after the pavement was completed This indicates that on this section the rate of infiltration of foreign material has been almost negligible.

A typical installation consisted of seven gauge points set in a longitudinal row on the quarter point of the pavement width The points were spaced approximately 10 in. apart in order to accommodate the measuring span of the special extensometer which was so graduated as to permit direct readings to be made to 0 0001 in. The seven points covered

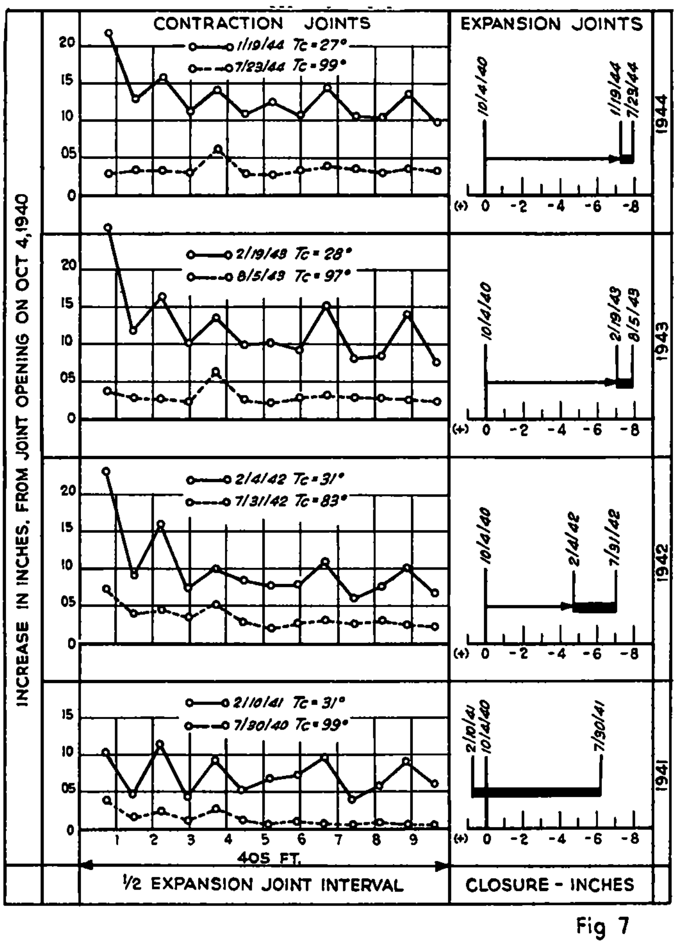


Figure 7. Annual & Permanent Changes in Joint Openings. Expansion Joint Interval = 810 ft. Contraction Joint Interval = 30 ft. (Base = Oct. 4, 1940)

LONGITUDINAL COMPRESSIVE STRESSES

During construction, special extensometer points were installed at several selected positions on the project with the thought they might prove useful at subsequent high temperature periods in determining the compressive stresses that might develop from year to year.

a length of pavement of approximately 60 in. The extensometer readings were referred to a standard "Invar" control bar at all times.

As a reference base in making these measurements, one installation of these points was placed immediately adjacent to a 1-in expansion joint which was one of a long series spaced at 120-ft. intervals. This point was selected in order to provide measurements of a section of pavement which was subjected to the least possible restraint

Other installations were placed as nearly as possible at the midpoints of various expansion joint intervals In all cases, the seven points were set in the interior of a panel so that no joint crossed the installation.

dition found at the quarter points These curves show that below a temperature of about 80 F available expansion space exists in the individual contraction joints and no restraint is present except subgrade frictional resistance acting against the expansion of the individual slabs From 80 F to about 90 F the joints are becoming tightly closed

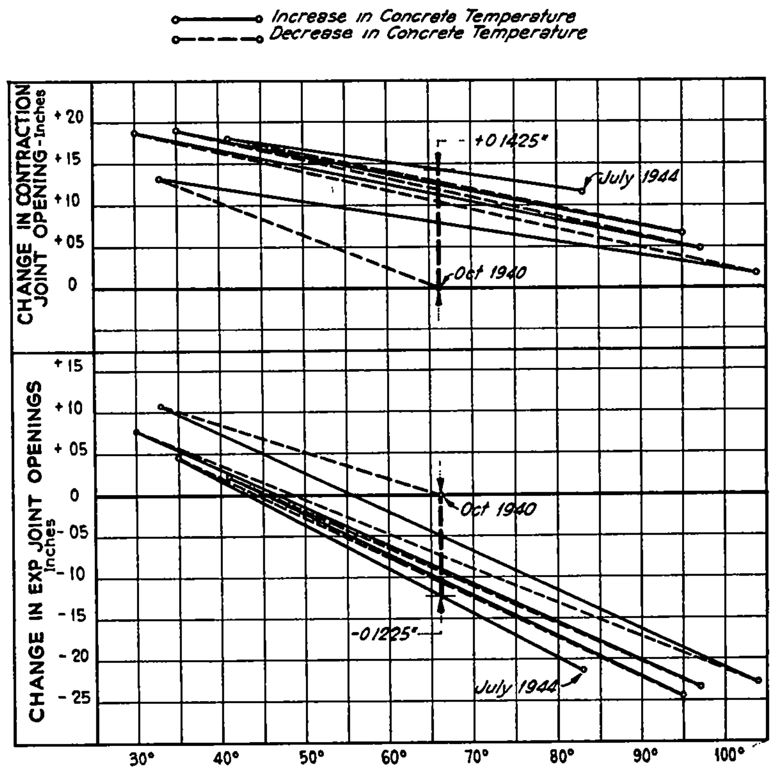


Figure 8. Annual & Permanent Changes in Joint Openings. Expansion Joint Interval = 120 ft. Contraction Joint Interval = 120 ft. Panel Length = 60 ft. (Reinforced Design)

As an illustration of the data obtained from these installations, Figure 10 shows the results of the July, 1944 measurements which were made at the center and the quarter points of Division 9 (one mile without expansion joints)

The straight line (No. 1) shows the length change-temperature relation at the reference station where very little restraint to expansion was present. Curve No 2 shows the relationship found at the center of this Division, while curves Nos. 3 and 4 represent the con- and restraint is built up. The curves show that full restraint to further expansion is developed at the two quarter points at about 90 F and at the midpoint at 95 F.

The thermal coefficient, 0 0000053, shown for the straight-line curve closely checks the laboratory determination of 0 00000545 made on this concrete in the same temperature range

The highest average slab temperature found on this project to date was 112 F

A close approximation of the stress caused by this temperature can be determined from these curves as indicated in the figure by the lettered intercepts, as follows: (1) Select some temperature value below the point of tangency of Curve No 2, say 70 F , and read the total unit change in length for both Curves Nos 1 and 2 from 70 deg to 112 deg These total unit changes are:

For Curve No 1	*For Curve No 2*
ac = 0 0000954	ab = 0 0000465
df = 0 0001272	ef = 0 0000170
Total = 0 0002226 = (X)	Total = 0.0000635 = (Y)

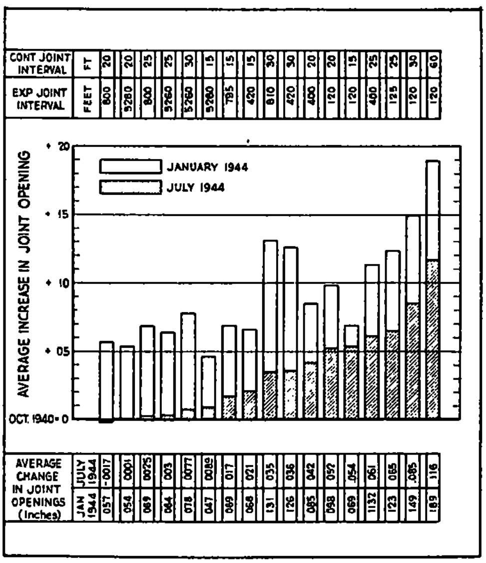


Figure 9. Average Increase in Contraction Joint Openings. From Oct. 1940 to Jan. & July 1944.

(2) Subtract (Y) from (X) 0 0002226 − 0.0000635 = 0 0001591 = (d) Then, (d) represents unit restrained expansion at the midpoint of this Division at a slab temperature of 112 F.

(3) From the stress-deformation relationship. $E = \frac{s}{d}$ or, s = Ed. the average value of "E" for this concrete as determined from 10 laboratory tests is 4,300,000 Then, the unit stress (s) is equal to:

s = Ed
or, s = 4,300,000 x 0 0001591
s = 684 lb per sq in

Similar computations for the quarter points give the following stresses.

For Curve No 3, s = 658 lb per sq in
For Curve No. 4, s = 699 lb. per sq. in.

Theoretically, the same value of stress should have been given by these computations for all three points for a slab temperature of 112 deg , since all three points are in full restraint at that temperature The small

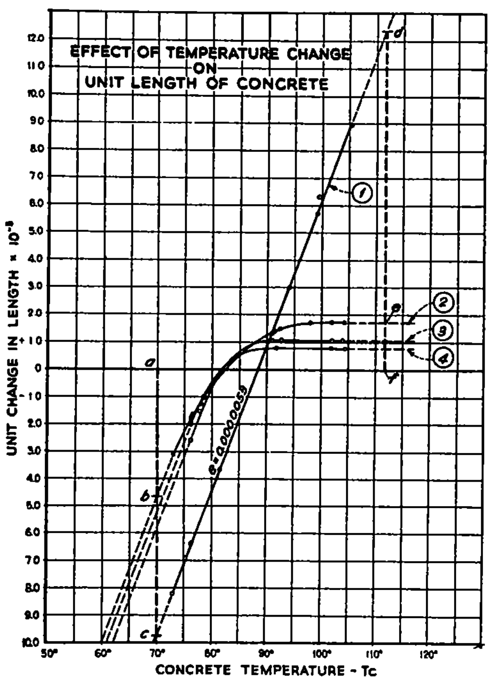


Figure 10

differences shown are due to small errors in drawing and reading the curves.

Since these measurements were made directly on the pavement on the same date and at four points, one of which permitted substantially free expansion, it is felt that they provide a close approximation of the actual stress existing in the pavement at that time The effect of moisture content of the concrete does not enter this determination of stress since the measurements were all made during the same 24-hr period, during which there had been no precipitation No precipitation occurred for a considerable time either prior to or after this 24-hr. period.

It would appear on the basis of the foregoing data that no serious compressive stresses have developed in this pavement up to the present time (4 yr after construction) The indicated stress is only about ¼ the ultimate compressive strength of this concrete.

SEASONAL MOISTURE CHANGE COEFFICIENT

For use in determining changes in the moisture content of the concrete and the subgrade soil, a considerable number of gypsum blocks were installed during the construction. These blocks and the method of moisture determination in which they are used was developed by Dr G J Bouyoucos and are described in Technical Bulletin No. 172 of the Michigan State College, April, 1939

The data obtained by this method appear to be consistent and indicative of a general but slow increase in moisture in the subgrade. These data, however, have not been analyzed in detail to date. Where the blocks were installed in the concrete, the results have not been satisfactory

While quantitative data on seasonal changes in moisture content of the concrete have not been obtained by the use of the Bouyoucos method, a series of special extensometer measurements has yielded data on the effect of these changes as indicated by corresponding changes in the measured length of a section of pavement slab.

This is the same section that was used as a reference base for the stress determinations described in the preceding section of this report. The seven gauge points covered a slab length of approximately 60 in., and were set as closely as possible to an expansion joint in order to eliminate all possible restraint. This 60-in. length of slab is free of any joints or cracks, so that the measured lengths represent actual lengths of the concrete only at the times the measurements were made.

Measurements have been made over this section twice a year, winter and summer, since the winter of 1941–1942. The data secured up to, and including, the present summer (August, 1945) are shown in Table 5 Briefly, the procedure used in these computations is as follows.

1 The seasonal changes in length of the section and the seasonal range in mean slab temperature are computed, (Columns E and F, Table 5).

2 Using the seasonal range in mean temperature and a value of "e" (thermal coefficient) adjusted to the midpoint of this seasonal range, the theoretical thermal change in length of the section is computed (Column H). The "e" correction curve shown in Figure 11 was determined for this concrete by a series of laboratory tests.

3 The seasonal differences between the theoretical thermal change and the measured change in length are computed (Column I)

4 The unit seasonal change in length is then computed by dividing the values in Column I by those in the preceding line of Column B

5 Finally, the average unit length change or moisture change coefficient is computed by taking an average of the values in Column J.

As shown at the bottom of Column J, Table 5, the average value of this seasonal moisture change coefficient has been found to be 0 000118 It is to be noted that the values during 1943 were considerably below average. An explanation of this may be furnished by reference to Figure 2, which shows that from the fall of 1942 through the spring of 1943, the precipitation in this area was considerably below normal. This may have resulted in a lower than normal moisture content in the concrete.

This seasonal change in length due to moisture change is opposite and compensatory to the seasonal change caused by the seasonal change in temperature. For this concrete it compensates for a rise in temperature of approximately 20 F, (0 000118 ÷ 0.000006)

In addition to this compensatory change in length due to seasonal changes in moisture, there was a further shortening or shrinkage of the concrete during the early hardening period. Assuming a conservative value of 0 0001[2] for this factor, the initial hardening shrinkage provides a further compensation equivalent to the expansion caused by a 17 F, rise in temperature (0 0001 ÷ 0 000006)

The effect of this initial hardening shrinkage and the seasonal changes of moisture in the concrete is to relieve the stress which would be caused by restrained expansion due to a temperature rise of 20 F + 17 F, or 37 F

[2] "Reinforced Concrete Construction," By Hool, 2nd Edition 1917

In other words, concrete placed at a given temperature would not be subjected to expansion stress until its temperature reached a value of 37 F, above the placement temperature.

straint is reached in the neighborhood of 90 F., which checks rather closely the critical value of 91 F, (54 F. + 37 F.) computed on the basis of the shrinkage and seasonal moisture change coefficients above

LENGTH CHANGE COEFFICIENT
CAUSED BY SEASONAL CHANGES IN CONCRETE MOISTURE

FIG 11
TEMPERATURE CORRECTION FOR THERMAL COEFFICIENT "e"

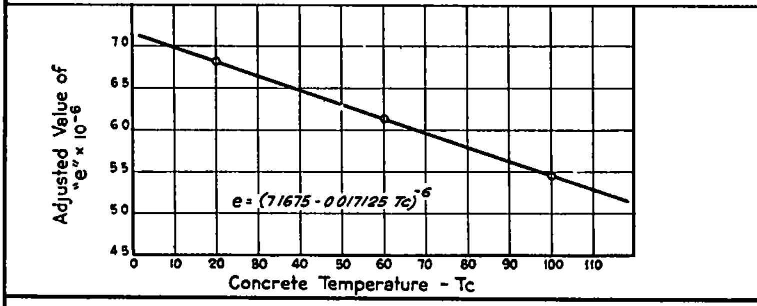


TABLE 5
MEASUREMENT DATA & COMPUTATIONS

DATE	MEASURED LENGTH OF CONCRETE	TIME	MEAN TEMPERATURE OF SLAB	SEASONAL CHANGE IN MEASURED LENGTH	SEASONAL CHANGE IN MEAN TEMPERATURE	ADJUSTED THERMAL COEFFICIENT "e" (1)	THEORETICAL THERMAL CHANGE IN LENGTH	DIFFERENCE IN LENGTH (H) - (E)	UNIT LENGTH CHANGE DUE TO MOISTURE CHANGE (I)÷(B)
(A)	(B)	(C)	(D)	(E)	(F)	(G)	(H)	(I)	(J)
2/11/41	59 9831"	2 07 PM	30.2°	–	–	–	–	–	–
7/30/41	59 9991"	4 28 PM	105 1°	+ 0160"	+ 74 9°	6 0090^{-6}	+ 0270"	0110"	00018338
2/5/42	59 9820"	2 08 PM	37 4°	- 0171"	- 67 7°	5 9473^{-6}	- 0242"	0071"	00011834
8/1/42	59 9967"	1 58 PM	96.2°	+ 0147"	+ 58 8°	6.0235^{-6}	+ 0212"	0065"	00010837
2/20/43	59 9853"	1 20 PM	43 1°	- 0114"	- 53 1°	5 9747^{-6}	- 0190"	0076"	00012667
8/7/43	59 9997"	11 47 AM	92 0°	+ 0144"	+ 48 9°	6 0107^{-6}	+ 0176"	0032"	00005335
1/19/44	59 9860"	4 37 PM	38 0°	- 0137"	- 54 0°	6 0544^{-6}	- 0196"	0059"	00009833
7/25/44	59 9982"	3 29 PM	88 7°	+ 0122"	+ 50 7°	6 0826^{-6}	+ 0185"	0063"	00010502
2/22/45	59 9834"	7 26 AM	23 5°	- 0148"	- 65 2°	6 2068^{-6}	- 0243"	0095"	00015834
8/2/45	60 0031"	6 27 PM	94 7°	+ 0197"	+ 71 2°	6 1554^{-6}	+ 0263"	0066"	00011086
Average Value of Moisture Change Coefficient →									00011807

(1) Corrected to middle of Seasonal Temperature range, from adjustment curve in Fig 11 above

This is illustrated by reference to Figure 10, which shows temperature-expansion curves based on measurements made at the center and quarter points on Division 9, which was a full mile without expansion joints. The concrete at this point was placed at a temperature of 54 F, and the curves show full re-

DAILY TEMPERATURE DIFFERENTIALS IN PAVEMENT

For use in conjunction with the extensometer measurements, a series of thermocouples was installed at three different locations on this project. The detail of these installations was shown in the 1940 report as published in Vol 20 of the Proceedings

At each of the three locations, two sets of six thermocouples were placed vertically in the pavement, one set within a foot of the pavement center line and the other set about 18 in. from the edge. The upper thermocouple was approximately ½ in. below the pavement surface and the lower one was about ½ in. above the bottom of the slab Between these, the other four thermocouples were spaced at intervals of approximately 1 in.

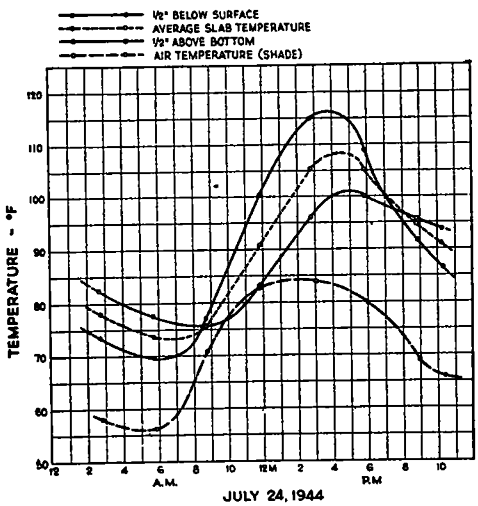


Figure 12. Typical Daily Change in Air and Concrete Temperatures, July 24, 1944. Payment Section: 9–6–9. Temperatures Taken 6″ from Centerline of Pavement.

Temperature measurements were made at these points every day, and over the same period that other measurements were made, in order to provide a continuous temperature control curve for the day's measurements

Data secured from one of these temperature control stations on a typical day are shown on Figure 12. These measurements were made on July 24, 1944 from 2:50 A M until 10 22 P M. at about 3 hr intervals The curves in this figure are self-explanatory and are typical of the conditions that exist on a typical sunny summer day They show the two reversals in temperature between top and bottom of the slab and the lag between air and concrete temperature that occurs under these conditions.

The maximum differential in temperature that developed on this particular day occurred at about 3·30 P M and amounted to about 17 deg. or nearly 3 deg per inch of slab thickness

The maximum differentials that have been observed on this project were 21 deg at 1 30 P M on August 1, 1942 and 23 2 deg at 3:00 P.M. on August 7, 1943. On the latter date, the differential approached closely to 4 deg per inch

It should be borne in mind that the upper and lower thermo couples were both embedded approximately ½ in. in the slab, so that the differentials given are smaller than those that actually exist between the extreme upper and lower fibers of the slab.

In the winter, the differential is usually less than 1 deg per inch and seldom exceeds that value.

PRESENT CONDITION OF THE PAVEMENT

The general condition of this experimental pavement in August, 1944, was good There have developed no apparent differences in condition or performance between the sections of 7-in uniform thickness and the equivalent 9-6-9-in thickened edge sections As pointed out in more detail later, a few of the non-reinforced panels have developed typical transverse cracks due to temperature stresses, while none of the longer reinforced panels have developed this type of failure.

It has not been possible to make periodic checks on the smoothness of the pavement using the precise level points which were originally installed for this purpose due to general damage to these points during the winter of 1940-41 by ice removal operations However, this property of the pavement has been checked on three dates by means of the Department's Road Roughness Recorder with the following results:

November 18, 1941	85 in per mile
February 3, 1942	84 in per mile
July 26, 1944	96 in per mile

These road roughness tests indicate only a slight increase in general roughness to date. Under the schedule of ratings which has been developed for use with this machine, this pavement would be rated as good or better than average after 4 years of service.

In January and July, 1944, string measurements were made on 1140 joints in this pavement for the purpose of determining the magnitude and extent of vertical deformation at transverse joints and the effect of panel length of joint deformation of this type These measurements covered more than half the joints on the project The average wintertime increase in vertical deformation at joints spaced at various intervals and therefore the effect of panel length on wintertime roughness of the pavement in comparison with summertime roughness are listed as follows

Panel Length	Increase per Mile	Av Increase in Def per Joint
ft	*in*	*in*
15	14 4	.041
20	25 6	097
25	35 4	.168
30	37.4	213
60	46 2	.525

In contrast with the roughness values, in inches per mile, secured with the Road Roughness Recorder, the above values are absolute values and represent the increase in elevation of the joints from summer to winter with respect to the midpoint of adjacent panels, whereas, the Recorder values cover all types of roughness in the pavement surface and represent the accumulated spring deflections of the unit at the time of the survey

The foregoing values of joint deformation indicate the beneficial effect of short panels in reduction of wintertime roughness due to high joints.

A transverse crack survey made on July 26, 1944, disclosed a total of 66 such cracks Of these, 30 were accounted for as being caused by localized subgrade disturbances The other 36 were distributed among the various non-reinforced panel lengths as follows

Panel Length	Total No of Panels on Job	No of Cracked Panels	Panels Cracked
ft			%
15	626	3	0 48
20	465	4	0 86
25	315	9	2 86
30	313	20	6 39
Totals	1,719	36	

None of the 48 60-ft reinforced panels contain transverse cracks caused by temperature stresses.

These data again indicates the advantage of relatively short spacing of contraction joints by reason of the reduction in the amount of transverse cracking.

Only three typical exterior corner breaks have developed to date No logical or consistent explanation of the cause of these breaks is apparent

Five faulted transverse joints were found on the entire project during the July, 1944, survey These joints, together with pertinent data, are listed as in Table 6

TABLE 6

Station	Pavement Section	Type of Joint	Dowels Used	Adj Panel Lengths	Dist to Nearest Exp Jt
				ft	*ft*
219 + 26 6	9-6-9-in	Keyed Const	No	30-30	514
275 + 85 6	7-in	" "	No	30-25	62
317 + 23 7	9-6-9-in	" "	No	20-30	29
573 + 23 1	7-in	Expansion	No	30-30	0
569 + 63 3	7-in	"	No	15-15	0

The faulted joints all had the same characteristic in that in every case the high side of the joint was the side from which the traffic approached the joint The high and low sides of the joints were reversed on opposite sides of the centerline, the differential in elevation being zero at the centerline and a maximum at the edges The differential about 2 ft from the edges was about ¼ in

Measurements of these joints made in March, 1944, indicated a somewhat more pronounced degree of faulting. At that time the maximum differential was ⅜ in

There has been no evidence of "pumping" at these joints or at any other location on this project

It may be observed from Table 6 that none of these faulted joints contained dowels, four of them were at or very close to expansion joints, and the one most distant from an expansion joint is located between 30-ft. panels It has been noted throughout this project that the keyed-type construction joints have exhibited a marked tendency to open up wider in cold weather than the conventional dummy-type contraction joint.

INDICATIONS AT THE PRESENT TIME

Based on the service performance of this pavement and the experimental data secured to date, the indications at the present time are as follows:

1. Expansion joints do not appear to be necessary; in fact, their omission appears to be beneficial in preventing the development of undesirably wide openings in adjacent contraction joints.

2. Short panels, on the order of 15 ft. long, are desirable because they provide (1) a reduction in the width of individual joint openings, (2) a reduction in pavement roughness, (3) a reduction in warping stresses, and (4) a reduction in intermediate cracking.

3. The 7-in. uniform section and the 9-6-9-in. thickened-edge type, appear to be equally satisfactory so far as performance on this project is concerned

4. Load transfer dowels do not appear to be necessary in dummy-type contraction joints spaced at relatively short intervals where expansion joints are omitted. This is equally true of both types of pavement section, the thickened-edge as well as the uniform depth section.

5. There appears to be no advantage in the use of a reinforced design involving relatively long reinforced panels in comparison with a non-reinforced short panel design. Furthermore, such a design has certain physical disadvantages in service performance due to greater changes in joint openings

6 Contraction joints spaced at intervals greater than 15 ft. are very difficult to maintain in a sealed condition when the usual types of asphaltic sealing materials are used. Certain rubber-base materials are more successful, but these decrease in effectiveness as the panel length is increased.

7. Metallic water seals (copper) of several different designs have not proved effective on this project

8. The need for non-extrusive joint fillers increases as the interval between expansion joints is increased

INVESTIGATIONAL CONCRETE PAVEMENT IN MISSOURI

By F. V. Reagel, *Engineer of Materials, Missouri State Highway Department*

The Missouri project was described at the Twenty-first Annual Meeting of the Highway Research Board (1941)[1] and the proposed measurements and observations outlined. Following is a brief report covering the period from the date of construction, June 1941, through August 1944, adding some descriptive details omitted from the original report and giving a summary of the cross-joint measurements and the condition surveys.

CLIMATE

The climatic environment of this project is typical for northern Missouri Figure 1 shows the monthly average of the daily maximum and minimum temperatures for ten years from 1934 to 1943 inclusive; and also the 56-yr. average rainfall by months. It is worthy of note that there are no months of the year in which the average daily maximum temperature is below the freezing point, but there are four months when the average daily minimum temperature falls below this point. Such a condition results in a pavement being subjected to a large number of cycles of freezing and thawing

TRAFFIC

Volume of traffic on the project for the years 1941, 1942, and 1943 is shown as follows The figures were obtained by taking weighted averages of traffic counts at three stations One station was near the south end, one was near the center, and one was about three miles north of the north end of the project.

[1] F. V. Reagel, Investigational Concrete Pavement in Missouri, *Proceedings*, Highway Research Board, Vol. 21, p 150 (1941)