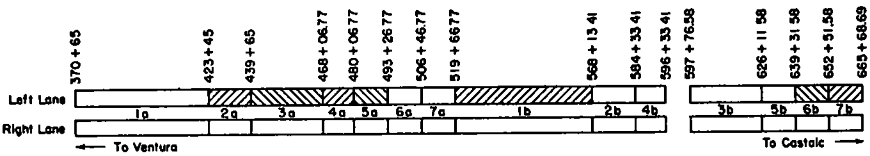
INVESTIGATIONAL CONCRETE PAVEMENT IN CALIFORNIA

By Thomas E Stanton, *Materials and Research Engineer, California Division of Highways*

The California project follows the scope outlined by E. F Kelley, Chief of the Division of Physical Research, Public Roads Administration,[1] and in general consists of the seven combinations of expansion and contraction joint spacings, load transfer bars, pavement cross sections and reinforcement shown in Figure 1 of this report.

DIAGRAM



▨ – Sections in which Joint and Level Measurement Points were placed

Section	Length		Cross Section	Expansion Joints		Weakened Plane Joints			Reinforcement
	Left Lane	Right Lane		Spacing	No. of Dowels	Spacing	Number of Dowels		
							Left Lane	Right Lane	
1a	5280'	5280'	9"-7"-9'	5280'	9	15'	0	4	None
1b	5280'	5280'	9"-7"-9"	5280'	9	15'	0	4	"
2a	1620	1620'	9"-7"-9"	810'	9	15'	0	4	"
2b	1620'	1620'	9"-7"-9"	810'	9	15'	0	4	
3a	2835	2835'	9"-7"-9"	405	9	15'	0	4	'
3b	2835	2835	9"-7'-9"	405'	9	15	0	4	'
4a	1200	1200	9"-7"-9"	120'	9	15'	0	4	
4b	1200'	1200'	9'-7"-9'	120	9	15	0	4	'
5a	1320'	1320'	9"-7"-9"	120'	9	15'	9	9	"
5b	1320'	1320	9 -7'-9"	120'	9	15'	9	9	None
6a	1320'	1320	9 -7 -9	120'	9	60'	9	9	Wire Mesh
6b	1320'	1320'	9'-7'-9'	120	9	60	9	9	Wire Mesh
7a	1320'	1320'	8 Uniform	120'	0	15'	0	0	None
7b	1320'	1320'	8 Uniform	120	0	15'	0	0	None

Figure 1. Diagram and Table Showing Locations of Various Types of Pavement Construction

As no previous reports have been published on the California project, this report includes the construction data as well as a summary of subsequent measurements and condition surveys covering an observation period of approximately four years from the construction in October 1941 to most recent measurements in August, 1945

LOCATION AND GENERAL DESCRIPTION

The concrete pavement project selected for the tests is located in the Santa Clara River Valley, Ventura County, about 20 miles inland from the Pacific Ocean. The elevation is from 675 to 900 ft. above sea level The geography of the surrounding country is such that the project is subjected to wide ranges of climatic conditions, not only from season to season and from day to day but also during different portions of the same day.

The test highway extends from the northeast end of the bridge over Piru Creek, to a short distance east of the Los Angeles County line (Fig. 2), the net length of the pavement being 5 65 miles.

There are duplicate sets of experimental sections, consisting of seven different types of construction designated as Sections "1a" "7a" and "1b" to "7b" respectively In order to avoid a break in the middle of an experimental section at a railroad crossing the positions of Sections 3b and 4b were reversed. Locations of the various types are given in Figure 1.

Although this pavement was constructed

[1] "History and Scope of Cooperative Studies of Joint Spacing in Concrete Pavements", *Proceedings*, Highway Research Board, Vol 20, p. 333 (1920)

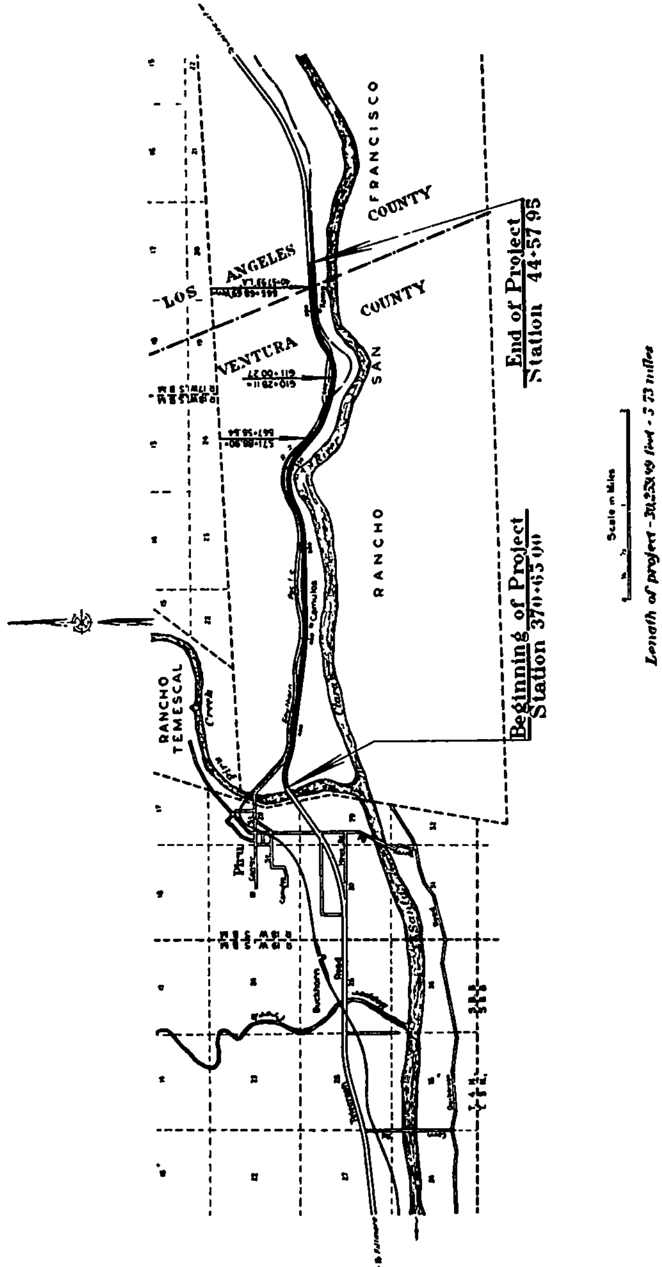


Figure 2. State Highway in Ventura and Los Angeles Counties, between Piru Creek and Los Angeles County Line

in October, 1941 the first measurements were not made until February, 1942 to allow time for initial cracking at the contraction joints.

Selection of Project. A review of the proposed highway on Route 79 in Ventura County between Piru Creek and Los Angeles County line disclosed that ample material of uniform quality was available for the construction of a blanket of selected subgrade material under a portland cement concrete pavement of suitable thickness for the purposes of the investigation and that the grades and curvature were not of a nature to prejudice the results.

reinforced sections 6a and 6b the slabs are 60 ft. long with expansion and weakened plane joints alternating.

Reinforcement. The reinforcing in sections 6a and 6b consists of wire mesh mats 10ft. 6 in. wide by 16 ft. long with No. 4 gage transverse wires at 12-in. centers and No. 0 gage longitudinal wires at 6-in. centers except that the outer four wires on each side are No. 00 gage wires at 6-in. centers.

The remaining sections are unreinforced.

Program. The program of observations and tests prepared by the Public Roads Administration was augmented, by measure-

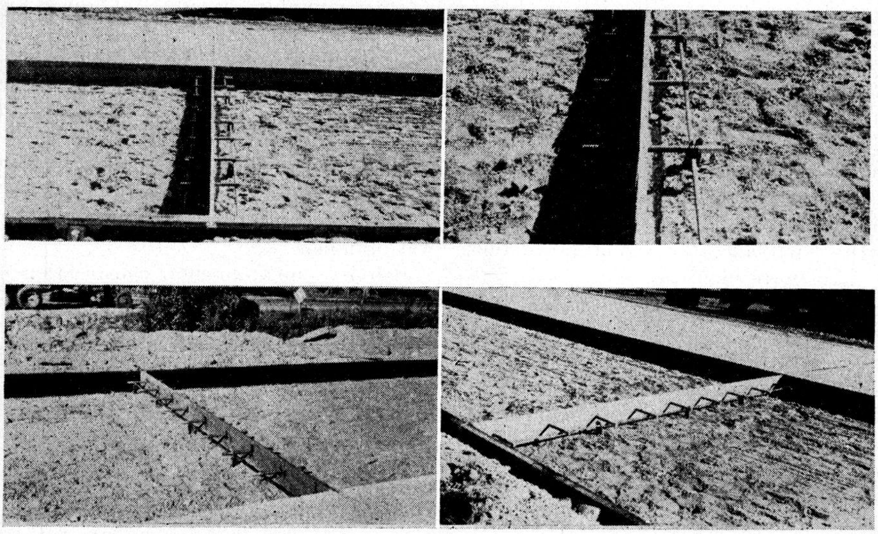


Figure 3. Expansion Joint Details. (Upper) Expansion Joint at Station 483 + 77; Alignment and Details. (Lower) Expansion Joint at Station 483 + 77; Redwood Joint Filler and Dowel Steel in Place.

Pavement Cross Section. The pavement has a 9:7:9-in. cross section with the exception of Sections 7a and 7b which are uniformly 8 in. thick.

Expansion Joint Filler. All expansion joint filler consists of $\frac{3}{4}$-in. redwood strips having an average weight of 22 lb. per cu. ft. oven dry and an average resistance of 1408 lb. per sq. in. to compression to one half the original thickness. Expansion joint details are shown on Figure 3.

Contraction Joints. Contraction joints are of the grooved dummy type and the spacing in the unreinforced sections is 15 ft. On the

ments of: (1) Temperature gradient within the slab, (2) Modulus of elasticity of the Concrete, (5) Coefficient of subgrade friction, (4) Overall movement of one lane of Section 1b, (5) Subgrade factor "*k*" and (6) Roughness measurements with the "Profilograph."

General. Grading and other preliminary operations were begun in 1940 and continued to early September, 1941. Concrete was placed in the right or south lane from September 20 to October 9, 1941. Paving of the left or north lane was begun October 10, and completed October 29.

No gage or level points were installed in the south lane. However, facilities for the special tests were installed in that lane during construction. These special installations were as follows:

1. Thermocouples for use with Micromax temperature recorders placed in the subgrade and in the pavement concrete during concreting operations. Temperatures within and below the slab were recorded during the hardening and curing of the concrete and at subsequent seasonal measurement periods.
2. Six detached pavement slabs, 15 ft. long for determination of subgrade friction. These slabs were separated from the adjoining pavement at each end by 4- by 9-in. removable timbers, set on edge.
5 Concrete disks for determination of Westergaard's "*k*" (subgrade factor). Twelve disks were installed as hereinafter described.

With the exception of the omission of the dowels and transverse bars at the dummy joints in Sections 1a to 4a and 1b to 4b of the north (left) lane, the structural details of both lanes were identical.

NOTES ON CONSTRUCTION

Grading. The natural soil in the excavations and the foundations under the fills, consisted for the most part, of a loamy soil with varying amounts of sand and silt. This material was classified as A-4, A-6, and A-7 according to the P.R.A classification. The California Bearing Ratio on the compacted and soaked material varied from 3 to 40 per cent at 0 1-in. penetration with the greater portion of the material falling in the range of 3 to 5 per cent.

Every effort was made to secure a thoroughly compacted foundation and subgrade which would not be subject to subsequent undue settlement or instability when saturated.

The top 12 in of the natural soil underlying all fills was compacted to at least 90 per cent relative compaction (California procedure) except for a few scattered locations. In the few cases where an excess of moisture was present the relative compaction dropped slightly below 90 per cent.

The material in roadway excavations and in foundations under embankment areas was, with a few exceptions, a poorly graded soil having a high silt and clay content with properties that rendered it extremely difficult to secure a uniform moisture condition during construction Such soils do not compact readily with equipment and methods in general use.

Uniform moisture content could be obtained only by adding water in small amounts and scarifying or cultivating after each addition of water during spreading operations and the further addition of water in small amounts during rolling operations. Not less than 20 passes, with sheepsfoot tampers were required to obtain the specified relative compaction This amount of rolling required twice the number of rollers specified as the minimum number of rollers under contract specifications.

It was agreed that the procedure of scarifying and rolling with double the minimum rolling equipment would be accepted as giving specified compaction This procedure was followed on fills constructed with materials hauled in "Euclids" and, with a few exceptions a relative compaction of 90 per cent or over was obtained.

Before embankment construction was started the natural ground under the proposed fills was thoroughly rolled. When the fill was less than 3 ft. high sufficient natural soil was removed to provide for a fill of 3 ft. and the entire fill was then constructed of sandy material obtained from channel change excavation.

The most suitable material found in the channel change was reserved for the upper 6 in. of the subgrade, material slightly less suitable was used as a subgrade blanket beneath this 6-in. upper layer. The least suitable material was placed in the lower portions of the embankments.

Moisture Control and Embankment and Subgrade Compaction:

The moisture in the excavated material frequently varied over a wide range during a given day's operations. As a consequence determination of the proper amount of added water for maximum density required constant checking and the exercise of considerable judgment

The moisture contents of embankments constructed with silty clay soils from roadway excavation ranged from 12 to 18 per cent with an average of approximately 15 per cent. A great majority of the tests showed a moisture content within 1 or 2 per cent of the average. In the few instances where less than 90 per cent relative compaction was obtained, the moisture content was too low for adequate compaction with the equipment used.

The moisture content in the embankments made with the sandy gravel from channel change excavation, ranged from 5 to 9 per cent with an average of approximately 8 per cent. The moisture content was much higher than this at the time the material was being compacted, but samples taken the day after the material had been placed showed that much of the excess moisture had evaporated or drained out.

Subgrade Uniformity. Uniformity of subgrade was obtained by selecting and blending material for the upper 6 in of the roadbed This selected material was placed in four or more layers, each layer being spread uniformly before placing the next layer. After sufficient loose material had been placed to provide for a 6-in compacted layer, it was mixed by power graders blading across the roadbed in successive windrows. The size of the windrows which could be handled was increased and the work expedited by the use of tractors to tow the power graders. A constant check was kept on the screen analysis of materials in the borrow pit and on the grade to assure uniformity. Screen analyses of the blended material from samples taken from the completed subgrade show considerable uniformity for the entire project All of the material placed in the upper 6 in of the subgrade classified as gravelly sand by grain size classification and as A-2 soil by P.R A. classification.

Because of its sandy nature and consequent low cementing value the subgrade material did not compact readily under a sheepsfoot tamper or conventional roller. However, under continued use by traffic and the contractor's equipment the material was ultimately compacted to a high density and became well cemented.

The subgrade immediately in advance of paving operations was scarified 3 to 4 in. deep, or just enough to remove the coarser rock and provide for shaping without materially affecting the density resulting from traffic.

Embankment Relative Compaction Tests. The relative compaction tests made on the material in place in the finished grade were based on the compacted density of the material as obtained by the standard California field method.

In general, relative compaction tests were made at 1-ft. to 2-ft. intervals as the fills were constructed, and if the compaction was found to be less than 90 per cent, further rolling or, where necessary, scarifying and re-rolling was required except in special cases

Subgrade Tests. After the layers of subgrade material had been mixed on the roadbed a number of samples were taken transversely across the full width. Tests on these samples showed exceptional uniformity. The extreme limits of grading of 75 per cent of the samples showed a variation of only 2 per cent for the material passing 100 mesh sieve and not more than 10 per cent for any of the coarser sizes. The analyses of samples from the finished subgrade showed slightly finer grading and slightly greater variation than the samples taken prior to compaction. This difference was probably due to breakdown of particles during construction and to the considerable amount of traffic to which portions of this layer were subjected before the pavement was placed.

Subgrade Moisture and Density Tests. Moisture and density samples were taken from the top 9 in. and 18 in. respectively of the subgrade.

Each sample was taken in two parts from adjacent holes 9 in. and 18 in deep

Subgrade density determinations consisted essentially in ascertaining the unit dry weight of the compacted material. They were made by excavating a hole, approximately 6 in in diameter, in the compacted subgrade to the desired depth, collecting and weighing all the material removed from the hole, determining its dry weight and measuring the volume of the hole by filling with sand of known unit weight. The ratio of dry weight of the soil removed to the weight of sand required to refill the hole multiplied by the known unit weight of the sand equals the unit dry weight of the soil in place.

The selected subgrade material contained some coarse particles grading up to 3- or 4-in. diameter. These coarse particles tended to drag for a short distance under the cutting edge of the Lewis subgrader causing holes or grooves in the finished subgrade. These depressions filled with concrete when the pavement was placed and as a result the bottom of the concrete contains numerous small projections into the subgrade.

Concrete. Data on the concrete materials and mixes are given in Table 1. The concrete strengths at early ages were apparently affected by the cement, a Type II cement with relatively low specific surface (1605) and low C_3S (as low as 32.0 per cent in some cases).

TABLE 1
DATA RELATING TO CONCRETE MATERIALS AND CONCRETE DESIGN
Ventura Test Project

(a) *Aggregates*
1. *Fine*—Granitic rock fragments, quartz and feldspars chiefly, with some biotite, hornblende and magnetite in the fine sizes. Specific gravity 2 70
Mortar strength 1 2 by volume—113 2% of Ottawa at 28 days
Loss in sodium sulphate test max 3 0%
2 *Coarse*—Chiefly granite and gneiss Specific gravity 2 70.
Loss in L. A Rattler at 500 rev —Max. 40 0 Loss in sodium sulphate test—max. 6 1%

(b) *Cement;*
Type II—90 8% passing 325 mesh—Specific surface by Wagner Turbidimeter 1605 to 1945 For the greater part 1600–1700
C_3A 3% to 6%—C_3S 32% to 54 3%.
Tensile Strength—3 days 210 to 345 psi, 28 days 410 to 475 psi.

(c) *Concrete*
Five sacks cement per cubic yard.
Water-cement ratio by weight 0 6. Slump 1½ in
Concrete mix by weight 1 2 73 3 66
The maximum size of coarse aggregate in the mix was 2½ in to 3 in , but in fabricating test specimens, all particles over 1½ in were removed
Average weight of concrete per cubic foot. Cyls 150 lb , Beams 152 lb , Cores 154 lb.

While the 28-day strengths were comparatively low the strengths at all later ages were up to normal and even above normal at 6 months. The following average figures for the entire job are of interest:

	Ave Compressive Strength, psi	
	Cylinders	Cores
4 days	1,210	
10 days	2,050	
28 days	3,610	4,240
3 months	5,210	
6 months	6,010	
1 year	6,570	
2 years		7,135

Subsequent compressive tests on 6-in. by 12-in. cylinders fabricated in the laboratory using the same cement and aggregate averaged over 7000 psi at one year. Tests of the mixing water showed it to be of excellent quality.

Strength test data are shown on Figure 4.

Joint Seal. For sealing the expansion and weakened plane joints a mixture of 20–30 air blown and SC-1 liquid asphalt was used.

Joint seal of the asphalt latex type developed by this department in 1936 was not used because neither natural nor synthetic rubber latex was available due to war restrictions.

CONCRETE CONSTRUCTION PROCEDURE

For the most part, standard equipment was used and standard procedure was followed during construction of the concrete pavement. Space does not permit nor is there any necessity for a detailed description of all construction equipment and operations except to state that the proportioning plant was an automatic batching plant. The concrete was homogeneous throughout the job and the workability good except for a few short periods when there was a slight tendency to harshness

The slump averaged 1½ in. with a range of ¾ in. minimum and 3 in. maximum. For the most part, however, the slump was between 1½ and 2 in.

Neither an excess nor a deficiency of mortar was noticeable and, although occasionally a small amount of pea gravel worked to the surface during finishing operations, no segregation was observed

Some bleeding was noticeable throughout the job At times it was barely perceptible but was generally pronounced although never excessive No washing away of cement was observed.

The time required for the hardening of the freshly placed concrete varied considerably with variations in temperature, humidity, etc.

As soon as the concrete had hardened sufficiently to permit walking on the surface, the expansion and weakened plane joints were filled with the asphaltic crack filler.

Curing. Concrete was cured by an earth cover kept moist by sprinkling for eight days. The dirt was spread by hand immediately behind the bump cutting operations.

The earth cover varied from ½ to 1 in. or more in thickness with an average of about ½ in. It was moistened at frequent intervals by means of a water distributor equipped with an extension side spray bar. Applications were frequent enough to prevent complete drying of the cover at any time. At the end of the eight day curing period the earth cover was removed with a motor grader.

Rainfall Record. None of the Ventura or Los Angeles County Official weather observation stations were near enough to the project to have similar climatic conditions Therefore, arrangements were made to secure rainfall data from an unofficial local rain gauge. Rainfall records thus secured show that 6.60 in. of rain fell between October 12 and December 31, 1941. The rainfall for the entire year of 1942 amounted to only 7.55 in. but in 1943 the total was 32.93 in. See Table 2.

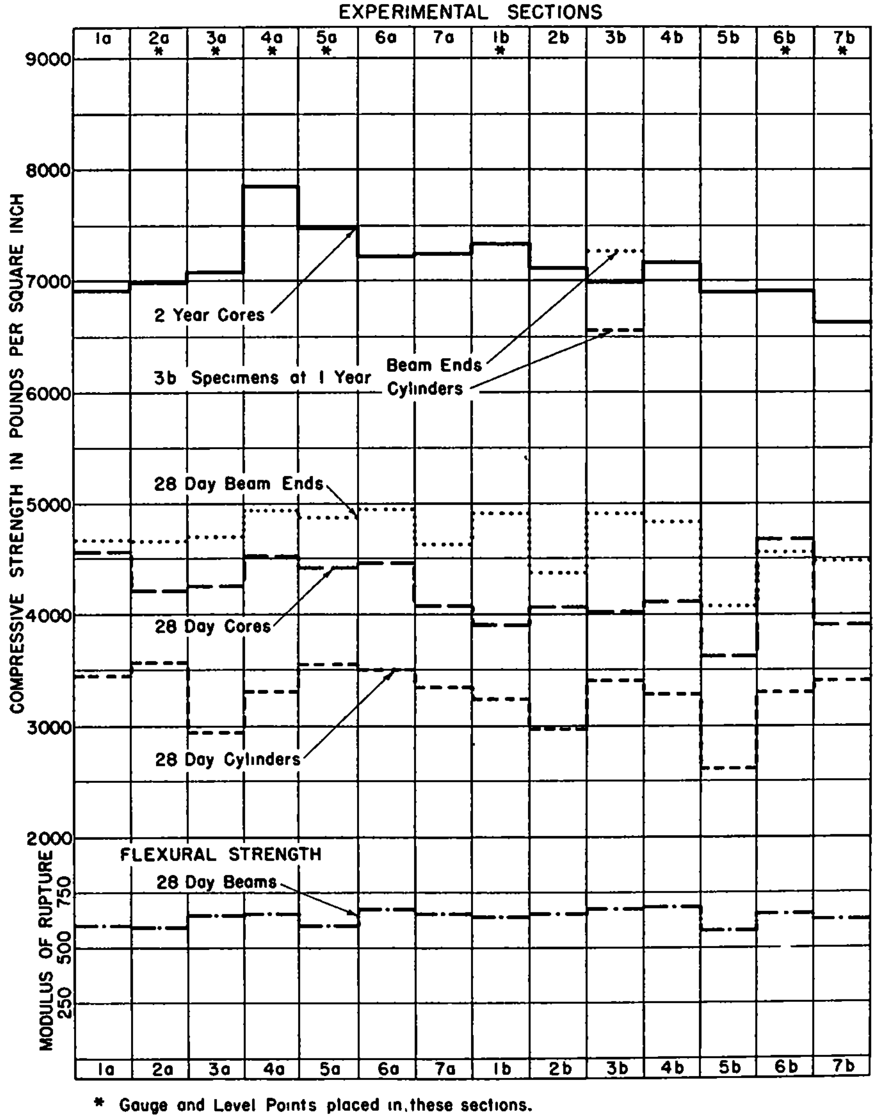


Figure 4. Average Compressive and Flexural Strengths of Cylinders, Cores, and Beams

DAILY OBSERVATION DURING CONSTRUCTION

Weather, Temperature, etc For temperature records a 6-point micromax recorder was used. Iron-Constantan thermocouples were placed at the center of the pavement lane as follows: Point No. 1, in the subgrade 6 in. below the base of the concrete slab; Point No. 2, in the bottom 1 in., Point No. 3, in the center, and Point No. 4, in the top 1 in. of the slab, Point No 5, air temperature, Point No. 6, wet bulb temperature. The wet bulb and air temperature were measured under a sun shield attached to the recorder box about 3 ft above the ground. The wet bulb temperature was secured by replacing the wet bulb thermometer of a Masons hydrometer with a thermocouple and depending on the natural circulation of the air for the evaporation of water from the hygrometer wick. The wet bulb temperatures are not plotted on the graphs, but were used to calculate relative humidity percentage.

TABLE 2
RAINFALL FROM OCTOBER, 1941 TO AUGUST, 1945

Month	Inches				
	1941	1942	1943	1944	1945
Jan		0 29	16 11	1 46	0 00
Feb		0 80	2 08	10 52	4 50
Mar		1 33	6 29	3 21	4 08
Apr		3 05	0 33	0 47	0 69
May		0 00	0 00	0 00	0 00
June		0 C0	0 00	0 00	0 00
July		0 00	0 00	0 00	0 00
Aug		0 12	0 00	0 00	1 26[a]
Sept		0 00	0 00	0 00	
Oct	1 33	0 82	0 00	0 27	
Nov	0 45	0 18	0 20	4 51	
Dec	4 82	0 96	7 92	1 07	
Totals	6 60	7.55	32 93	21 51	

[a] Rainfall during a single storm of approximately 1 hr 30 min duration This storm occurred one week before the August 1945 measurements

Sunshine with occasional clouds prevailed from October 10th to 18th during the pouring of Sections 1a to 6a and a portion of 7a

Light rain and clouds with occasional sunshine prevailed from October 19 to October 25, during the pouring of Sections 7a, 1b, 2b, 3b, 4b, and a portion of 5b No concrete was placed on October 26th and 27th.

Sunshine with occasional clouds again prevailed when the concrete was poured in Sections 5b, 6b, and 7b.

Temperature records are given on Figures 5 and 6, and Table 3.

Hours of Work with Stationing. A close record was kept of the time at which concrete was mixed and placed at each station as well as of any delays, etc., at intermediate points. The number of batches, volume of pavement slab, starting and quitting time for the mixer, the net operating time, etc. were recorded.

Details of Concrete Mixes. At the start of paving operations a few of the daily control samples of the fine aggregate showed a grading slightly below the specification limits. To correct this deficiency and in the hope of affecting a reduction in the bleeding, approximately 250 lb. of blending sand per batch (5 per cent of total aggregate weight) was used to replace an equivalent amount of the concrete sand.

The change from 37 per cent concrete sand to 32 per cent plus 5 per cent blending sand was first made on October 6th. On October 9, the blending sand was omitted for a portion of that day's run after which the use of the blended sand was resumed Otherwise, only minor variations in the proportioning of the aggregates were made throughout the paving of both lanes.

Concrete Temperatures. Supplementary to other temperature records notations were made of the temperatures of the freshly deposited concrete at various times during each day's work. Similar observations were made at the time of final floating See Table 3.

In addition special thermometer wells, designed to accommodate Western dial type thermometers were placed in the fresh concrete, 2 ft. from the left outside edge of the pavement at a number of points. Each installation consisted of two thermometers, one to determine the temperature of the top 2 in of the pavement, the other the temperature 5 to 6 in. below the surface

Time Intervals Between Placement, Finishing, Curing, Etc Records were kept of the time at which concrete was poured at each section and for each of the successive operations.

Joints Three-quarter inch redwood boards were used in the expansion joints throughout the project and no instances of poor alignment or lack of support were noticed although a slight vertical displacement of the boards occurred in a few instances during the placing of concrete

All dowels in the test sections were painted, thoroughly greased and provided with caps. All the caps were in place when the concrete was poured.

Although the pavement was constructed in two lanes of equal width no tie bars were used across the center joint. The steel side forms

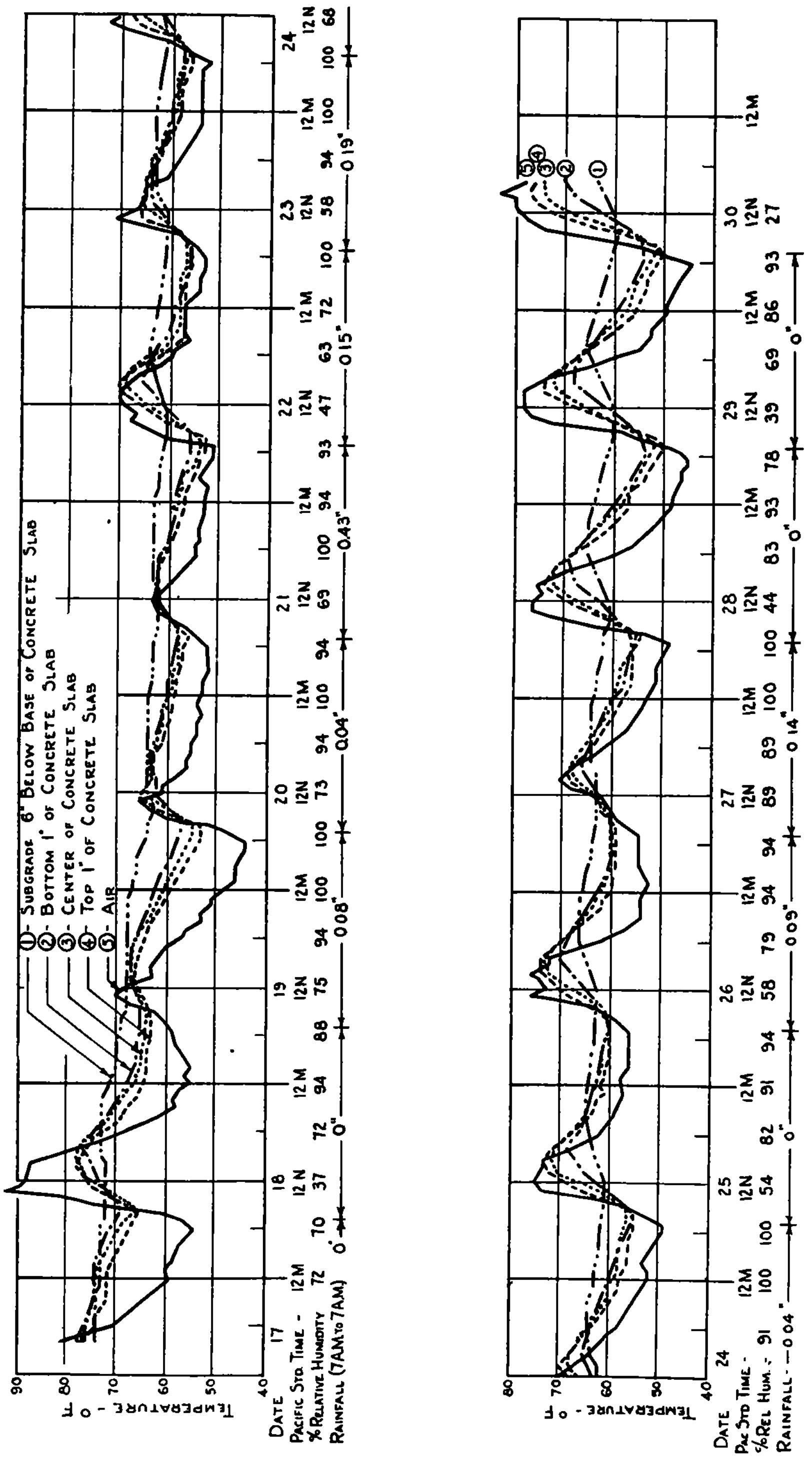


Figure 5. Temperature Record from October 17 to 30, 1941; 9:7:9 in. Slab Thickness, North Lane. Concrete Placed at 3:45 P.M. on October 17

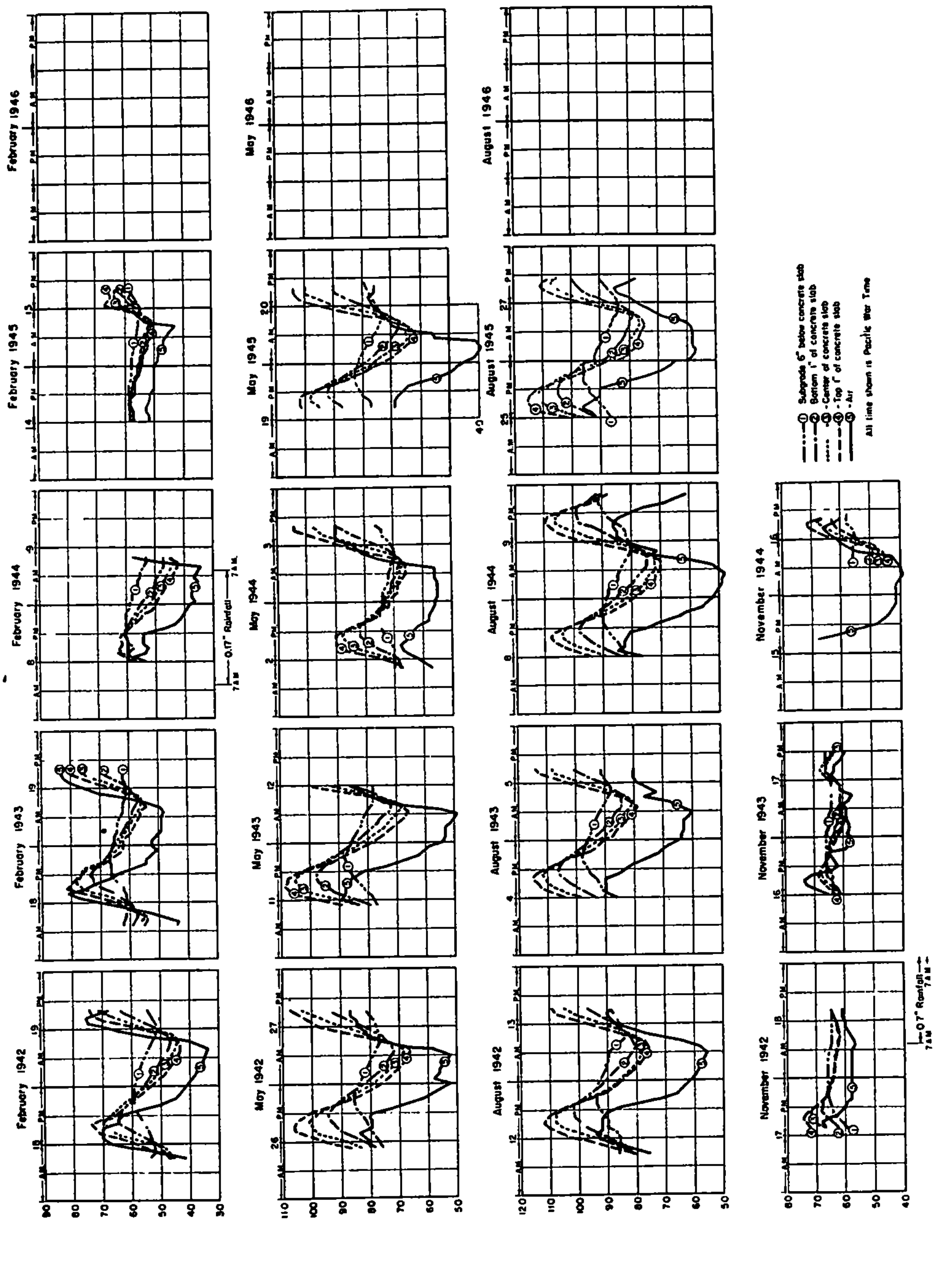


Figure 6. Daily Temperature Records; 9.7:9 in. Slab Thickness, North Lane

left a smooth surface on the inner edge of the concrete slab on the south lane throughout most of the project All rough spots or honeycomb were filled with cement mortar before the concrete for the second, or north, lane was poured.

The depth of the groove at the dummy joints was measured at frequent intervals for at least three points in a joint (near each edge and the center of the slab). Measurements were made after the concrete had begun to harden and before the groove was filled with asphalt. The depths varied from 2⅛ in. to 2⅝ in. and averaged approximately 2½ in.

TABLE 3
CONCRETE AND AIR TEMPERATURE RECORDS AT TIME OF POURING CONCRETE AND SIX HOURS LATER

Date	At Time Poured			6 Hours Later		
	Air	Concrete		Air	Concrete	
		Top	Bottom		Top	Bottom
10-15	84	70	70	87	76	74
10-16	62	67	67	91	82	72
10-18	52	65	65	80	76	70
10-20	46	66	66	59	67	67
10-24	52	61	61	65	72	68

The longitudinal center joint was a contact joint throughout the project. The use of the edging tools left a groove ¾ in. to 1½ in. deep. No regular series of depth measurements were made.

TEST SPECIMENS

Beam and cylinder test specimens were made each day on which concrete was poured Beams were given the same curing treatment as the concrete in the pavement The cylinders were cast in the molds and sealed with tight fitting covers immediately after fabrication.

Test data on cylinders and beams are given in Table 4.

When the concrete in each of the experimental sections (north lane) was 21 days old, cores were cut for 28-day compressive strength tests. In most instances the cores were cut at the locations from which the concrete for the test specimens had been obtained.

A similar set of cores was cut from the south lane in order to obtain a comparison of concrete strengths in the two lanes However, some of the cores from the south lane were older than 28 days (29 to 38 days) when tested. Corresponding cores were cut from the two lanes when the pavement was 2 years old.

While the 28 day tests indicated greater strength for the concrete without the blending sand, the 2-year cores indicated that the concrete in which the blending sand was used now equals or exceeds the compressive strength of unblended sand concrete.

The reason for the apparent superiority of the concrete with the unblended sand at the early ages may be due in part to the higher temperatures prevailing during the first part of the construction and curing period, during which time the unblended sand concrete was poured.

TABLE 4
COMPARISON OF COMPRESSION, MODULUS OF RUPTURE AND MODULUS OF ELASTICITY TESTS ON 36 FIELD SPECIMENS OF CONCRETE FROM AN EXPERIMENTAL PAVING PROJECT IN VENTURA COUNTY, CALIFORNIA
5 Sack Concrete

All figures are the average of tests on three specimens. Cylinders 6-in by 12-in Beams 6-in by 6-in by 34-in

	Compressive Strength		Mod of Rupture	Modulus of Elasticity $E \times 10^{-6}$			Secant E / Dynamic E 6- by 12-in Cyls
				Dynamic		Compr Cyls at 1000 lb psi	
	Cyls psi	Beam Ends psi		Beams	Cyls		
4 Days	1,210	2,140	333	3 47	3 46	1 19	0 34
10 "	2,050	3,320	588	4 29	4 47	2 31	0 52
28 "	3,610	5,120	678	4 92	4 90	3.06	0 62
3 Mo	5,210	6,670	797	5 61	5.38	3 89	0 72
6 Mo	6,010	6,390	859	6 00	6 03	4 58	0 76
1 Yr.	6,570	7,280	808	6 36	6 21	4 66	0 75

Note Dynamic determinations were made by measuring the fundamental frequency using "T" Correction Curve 4, Fig 1, and the same type equipment shown in Fig. 2, *Proceedings*, A S T M , 1941, Pages 1055 and 1059

Another plausible explanation is the fact that the C_3S content of the cement averaged somewhat lower on the latter part of the project than the first part.

Measurement Points. Points for measurement of joint openings and closings, wells for measurement of slab temperatures, elevation points and permanent bench marks were installed as provided in the agenda for the work.

TRAFFIC DATA

Truck data were secured through a special loadometer survey conducted at a point on the test highway on Tuesday and Wednesday, July 10 and 11, 1944.

The data on total traffic were compiled from traffic counts between 6 A M. and 10 P.M. on the middle Monday of each month from August 1943 to and including July 1944.

The traffic reaches a peak in July and August and falls off to a minimum in January and February.

Traffic data are given in Tables 5 and 6.

TABLE 5
CLASSIFICATION OF TRAFFIC 6 00 A M TO 10 00 P M MIDDLE MONDAY OF EACH MONTH[a]

Month	Number							
	Autos	Auto Trail	Busses	Trucks Light	Trucks Heavy	Comm Trail	Mil Veh	Total
1943								
August	1,042	30	11	213	248	86	33	1,663
September	985	8	10	179	302	73	8	1,565
October	914	21	12	132	236	60	10	1,385
November	873	13	12	162	204	63	11	1,338
December	840	20	13	110	168	49	21	1,221
1944								
January	769	13	14	126	168	61	30	1,181
February	761	15	10	134	167	65	12	1,164
March	725	19	11	176	277	67	3	1,278
April	841	14	18	123	234	58	9	1,297
May	927	20	20	154	270	76	18	1,505
June	955	11	9	194	261	99	1	1,530
July	1,010	27	12	158	352	112	6	1,677

[a] Traffic counts on LA-79-A at Castaic Junction

TABLE 6
NUMBER OF AXLE LOADS OF VARIOUS MAGNITUDES PER 100 LOADED AND EMPTY TRUCKS AND COMBINATIONS OF EACH TYPE WEIGHED ON JULY 10 AND 11, 1944

Axle Load in Pounds	Single-Unit Trucks	Tractor-Truck Semi-Trailers	Truck and Trailers	All Veh Types
Under 8,000	160 93	264 30	272 92	197 36
8,000– 8,499	2 65	25 00	29 18	11 01
8,500– 8,999	0 66	3 57	8 33	2 64
9,000– 9,499	2 65	10 71	20 83	7 49
9,500– 9,999	1 99	3 57	12 50	4 41
10,000–10,999	3 31	3 57	45 83	12 33
11,000–11,999	7 29	17 86	29 17	13 22
12,000–12,999	9 93	17 86	31 25	15 42
13,000–13,999	6 62	28 57	27 08	13 66
14,000–14,999	3 31	10 71	25 00	8 81
15,000–15,999	1 32	7 14	8 33	3 52
16,000–16,999		3 57	8 33	2 20
17,000–17,999	0.66	3.57	2 08	1 32
18,000–18,999				
19,000–19,999			2 08	1 32
Totals .	201 32	400 00	522.91	293 83

PERFORMANCE AND MEASUREMENTS SUBSEQUENT TO CONSTRUCTION

Joint Width Measurements

Average, seasonal and daily joint measurements to date are shown in Figures 7 and 8. On these figures it is possible to observe at a glance the pavement expansion and contraction fluctuations with the seasons as well as with the temperature variations

All joint width changes are based on the original joint width measurements in February, 1942 and therefore show the expansion or shrinkage of the concrete from that datum.

Seasonal Changes

On the average the maximum shrinkage of the concrete and hence largest joint opening at both weakened planes and expansion joints has occurred in November for all test sections.

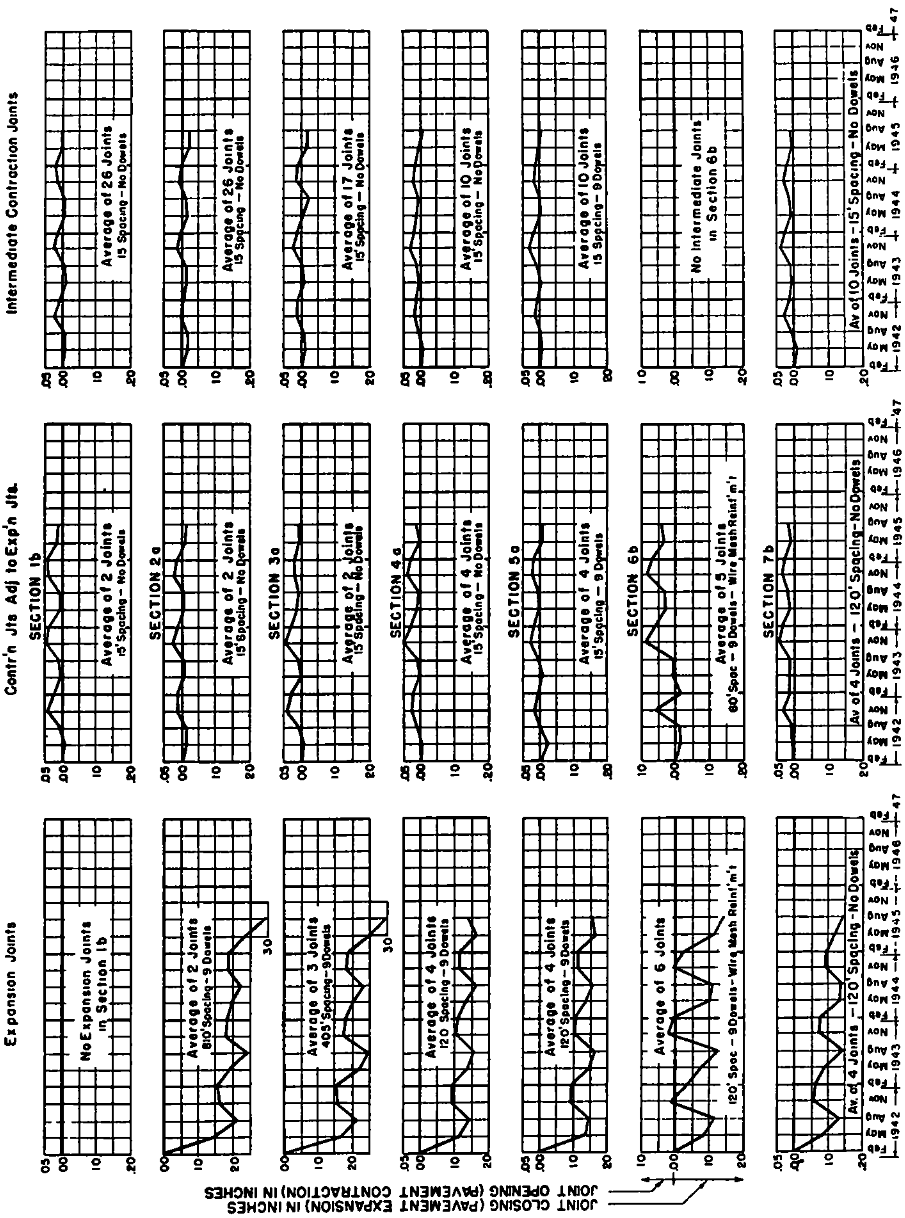
The expansion joints have measured the greatest concrete expansion in August, but in the case of the weakened planes the measurements indicate little difference on the average between May and August.

Daily Changes

The greatest daily fluctuation occurred in reinforced Section 6b where the expansion joints were spaced 120 ft with one intermediate weakened plane, the average movement amounting to as much as 0 1-in. at the expansion joints and 0.07-in. at the weakened planes in August 1943 as compared with 0.06-in. and 0.04-in respectively for an expansion joint spacing of 120 ft. with weakened planes every 15 ft. in the unreinforced section 5a.

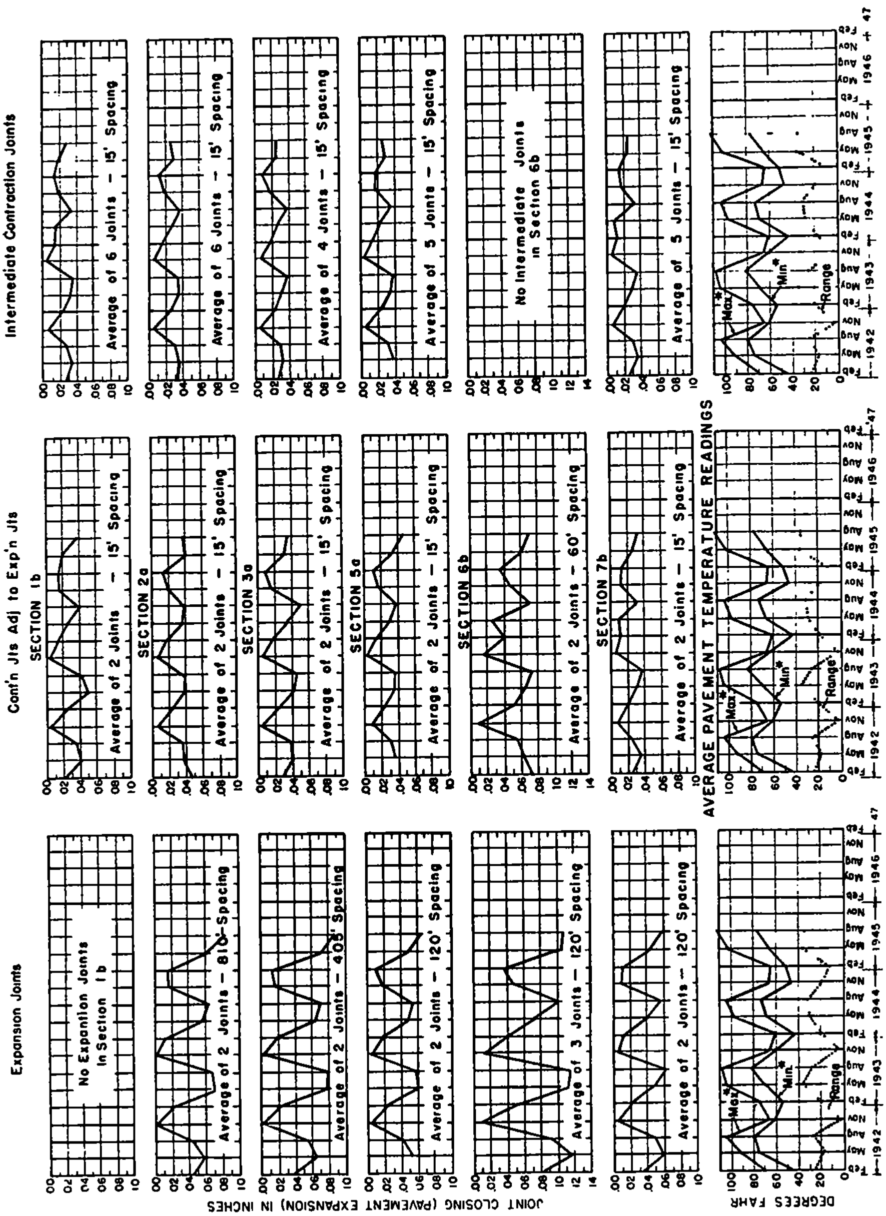
Permanent Changes

The permanent changes at the weakened planes with relation to the August, 1942 measurement are negligible to date (August, 1945). The changes at the expansion joints have been somewhat greater although in no case over 0 07 in. subsequent to August, 1942. The permanent change from the original construction, however, is somewhat greater averaging 0.16 in. for the 120-ft. spaced expansion joints and 0 30 in. for the 405-ft. and 810-ft. spacing



Note: Reinforcement used only in Section 6b

Figure 7. Average Change in Joint Widths, Seasonal



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

Figure 8. Average Change in Joint Widths, Daily

Elevation Measurements

Levels were taken at the elevation points in October, 1941 and again in February, 1943 and August, 1944.

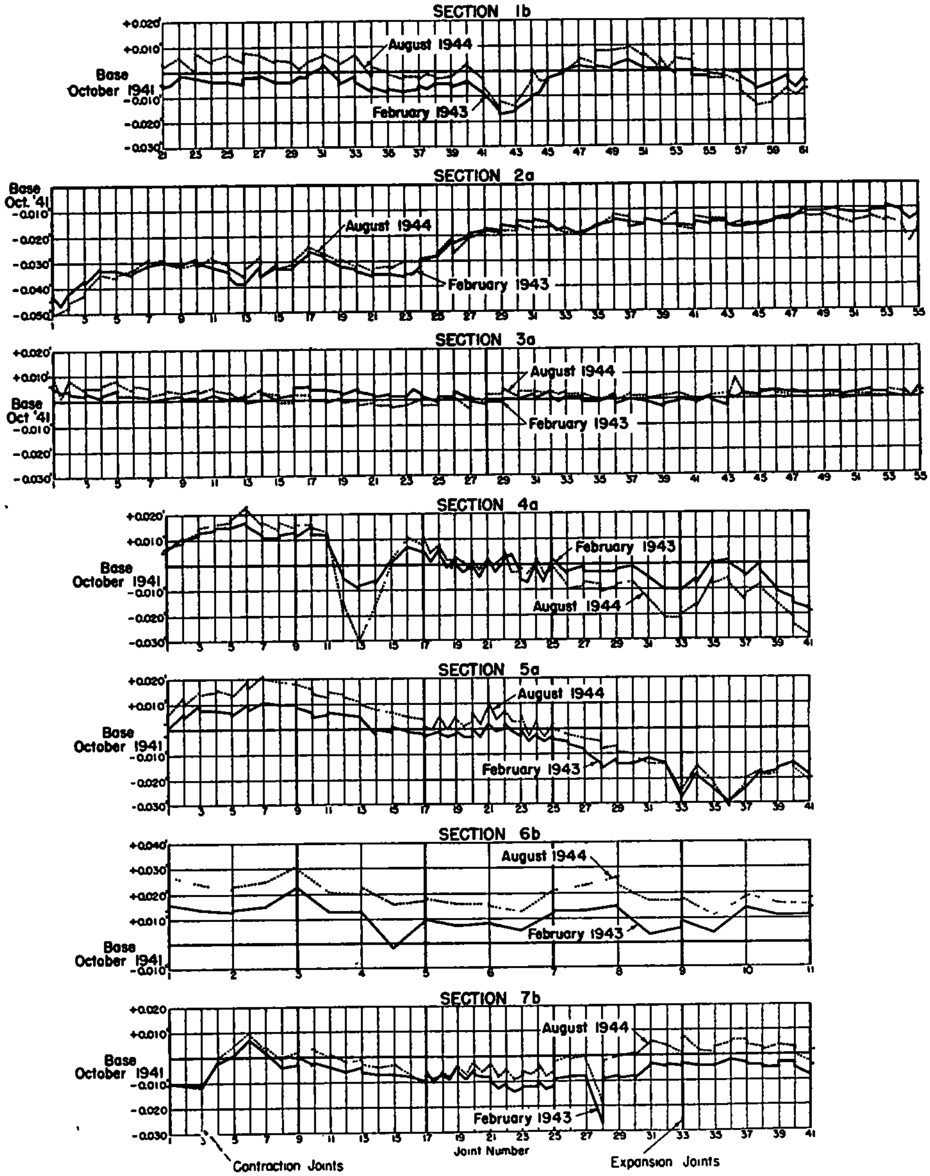


Figure 9. Ventura County Experimental Concrete Pavement Changes in Elevation

The relations of all elevation points in February, 1943 and August, 1944 to the original elevations of October, 1941 are shown in Figure 9. The vertical scale has been strongly exaggerated in order to show up very slight differential settlement at the joints. However, the maximum settlement since the initial level measurements in October, 1941 has been relatively slight, amounting to a maximum of less than 0.05 ft. This maximum settlement occurred at the west end of Section 2a where the embankment was between 7 and 8 ft high The next in order of settlement was at the east end of Section 5a with a fill of approximately 6 ft. The sharp sag at Joint 15 of Section 4a marks the crossing of an old drainage or irrigation ditch. Even so the total settlement is slight (0 03 ft.) although the fill on this section averages 6 to 8 ft.

The least vertical movement has been in Section 3a where the grading ranged from a slight cut to a maximum fill of approximately 3 ft The levels on several sections show a slightly higher elevation than in October, 1941 for a portion of the section, but only one section (6b; reinforced, 60-ft. between joints) is consistently higher than immediately after construction. The embankment on this section averages 3 ft.

It is of interest to note that slab center elevation points are as a rule at a relatively lower level than the joint points, thereby indicating definite warp at the joints It is to be regretted that the center elevation points were set at only a few places, usually in the central portion of each test section. However, as the level readings were supplemented by profiles run with the Profilograph, hereinafter described, there is a complete graphical record of the degree of warping at all joints for all sections

In analysing the variation in the extent of the warp as shown by the level readings consideration must be given to the time of day at which the readings were made, greatest warp occurring early in the morning when the surface of the concrete slab was at its lowest temperature and there was the greatest temperature differential between the top and bottom.

Crack Survey

A transverse crack developed in the north lane in experimental Section 3b at Sta 617+25 before the pavement was opened to traffic in November, 1941.

Three additional cracks had developed by August, 1944. Two are corner cracks in the southeast corners of the end slabs in each lane of the pavement (Sect. 7b Sta. 665+69). The other is a transverse crack in the north lane, test slab at the west end of Section 7a. The weakened plane joint which should have been constructed at this point was either accidentally omitted in both the north and the south lanes or the finisher failed to define the joint by edging The present transverse crack is at the position where the weakened plane joint should have been No visible crack has to date appeared at this point in the south lane

General Condition Survey

The surface of the concrete in August, 1945 at the end of nearly four years' service, shows practically no change from its appearance when first opened to traffic. All test sections are in excellent condition and no differences between any of the test sections in either lane can be detected by visual inspection.

In a few instances, there is some evidence of incipient faulting at expansion joints and at daily construction joints, but it is so slight that except for the level records it would be difficult to state with assurance that faulting has started.

There is no evidence of crushing, or of spalling due to movement of the concrete. At many points throughout both lanes, both at joints and the surface of the slabs, rocks and small pieces of concrete have become loosened and removed by traffic Present appearances as well as circumstances noted during construction, indicate injury during the finishing of the joints or the operation of the cut float. Hence any slight spalling to date under traffic can be traced to construction defects.

No evidence of change is noticeable in any of the redwood joint fillers. The top edges of a few, mainly those which were originally noted as high, have been exposed by removal of asphalt filler, under traffic

There is no evidence of "pumping" or of joint movement.

OVERALL MOVEMENT OF ONE LANE SECTION 1b

Concrete monuments for transit points were placed on each side of the pavement 1 ft below the ground surface, 3 ft. out from the edge, and 1 ft. back from each end of Section 1b for use as reference points to determine overall movement at the ends of this 1-mile test section. Copper tack transit points in lead were placed in the top of each monument and in the center of each pavement lane. These points were set in line January 14, 1942.

Table 7 shows the increase in length at each end of each lane of Section 1b from January 14, 1942 to August, 1945.

MODULUS OF ELASTICITY TESTS

Modulus of Elasticity determinations were made by compressive and dynamic (sonic) methods. One purpose of the tests was to determine the relation, if any, between the two methods.

The dynamic E tests were made on both 6- by 6- by 34-in beams and 6- by 12-in. cylinders; whereas, the secant stress-strain determinations were made on the cylinders only.

There was close correlation at all ages (4 days to 1 year) between the dynamic E determinations on both beams and cylinders, but the secant E averaged considerably less than the dynamic. See Table 4.

TABLE 7

Data	Overall Change in Length, Section 1b			
	W End (Sta 519 + 78 7)		E End (Sta 568 + 27 9)	
	Right Lane	Left Lane	Right Lane	Left Lane
	in	*in*	*in*	*in*
1-14-42	0	0	0	0
11-18-42	+0 03	+0 17	+0 10	+0 08
8- 4-43	+0 05	+0 20	+0 06	+0 12
11-16-43	+0 04	+0 20	+0 10	+0 12
2- 8-44	+0 00	+0 20	+0 08	+0 08
5- 3-44	+0 13	+0 15	+0 10	+0.17
8- 9-44	+0 13	+0 18	+0 08	+0 13
11-16-44	+0 06	+0 17	+0 12	+0 15
2-15-45	+0.12	+0 15	+0 10	+0 10
5-20-45	+0 15	+0 03	+0 20	+0 20
8-25-45	+0 10	+0 18	+0 17	+0 18

A special set of test specimens (eighteen 6- by 12-in. cylinders and eighteen 6- by 6- by 34-in. beams) was fabricated from the same batch of concrete mixed in the field in the job paving mixer during regular paving operations. The water-cement ratio and slump were therefore uniform for all specimens as well as the cement and aggregates.

The secant E of the 6- by 12-in. cylinders was found to be substantially the same at 1000 psi as at 25 per cent of the compressive strength for all ages from 28 days up.

In making the dynamic E determinations, the correction factor T described by Obert and Duvall in their paper published by A.S.T.M. in 1941[2] was applied, T correction curve 4, Figure 1 being used. Curve 4, which applies directly to prisms, was used for the 6- by 12-in. cylinders by entering the chart at an abscissa value which is $\sqrt{3/2}$ times the ratio of diameter to length For example for ratio of diameter to length of cylinder of 0.5, enter the chart at an abscissa value of 0 433.

Other Tests. Similar determinations were made at 28 days on other specimens fabricated throughout the life of the project.

Forty-eight cylinders and beams fabricated during this period tested as follows at 28 days.

Dynamic E (5- by 12-in) cylinders	4,800,000	
Dynamic E (6- by 6- by 34-in) beams	4,900,000	
Secant E (1000 lb) cylinders	3,000,000	= 62% Dynamic E
28 Day Compressive Strength (Cyls)	3,870 psi	
28 Day Compressive Strength (Cores)	4,240 psi	
28 Day Modulus of Rupture	639 psi	
5 0 sacks of cement per cubic yard		

It is of interest to note the close agreement of all dynamic and compressive E determinations on these overall job specimens with similar determinations on the 18 cylinder specimens from the single batch from which the special set of test specimens was fabricated.

The test results indicate some relationship between dynamic and secant E values. It appears that under conditions similar to those in the tests described in this report, the 28-day secant E at 1000 psi averages approximately 62 per cent of the dynamic E at the same age.

Tests on other projects, however, indicate that this factor is influenced by type and quantity of cement as well as by the water-cement ratio, grading and type of aggregates, etc. The secant E measurements on some other projects were approximately 80 per cent of the dynamic E at 28 days.

SUBGRADE FRICTION

Measurements of subgrade friction were made on full sized pavement slabs. The test slabs were separated from the adjoining

[2] Leonard Obert and Wilbur I Duvall, "Discussion of Dynamic Methods of Testing Concrete with Suggestions for Standardization," *Proceedings*, Am Soc Test. Mat'ls., Vol. 41, p. 1053 (1941).

slabs at each end by 4- by 10-in redwood block spacers, placed on edge during paving operations. These redwood spacers were subsequently removed, leaving a 4 in. clear opening at each end of the test slabs The inside edge of the slab was in contact with the adjacent slab of the other lane and the outside edge was in contact with the shoulder material.

measured with Ames dials attached to adjacent slabs at the opposite end to which the pressure was applied. The slab was then moved alternately east and west in approximate 0.2-in. movements until the force required became constant. This preliminary series of displacement movements was made to put the subgrade in somewhat the same condition

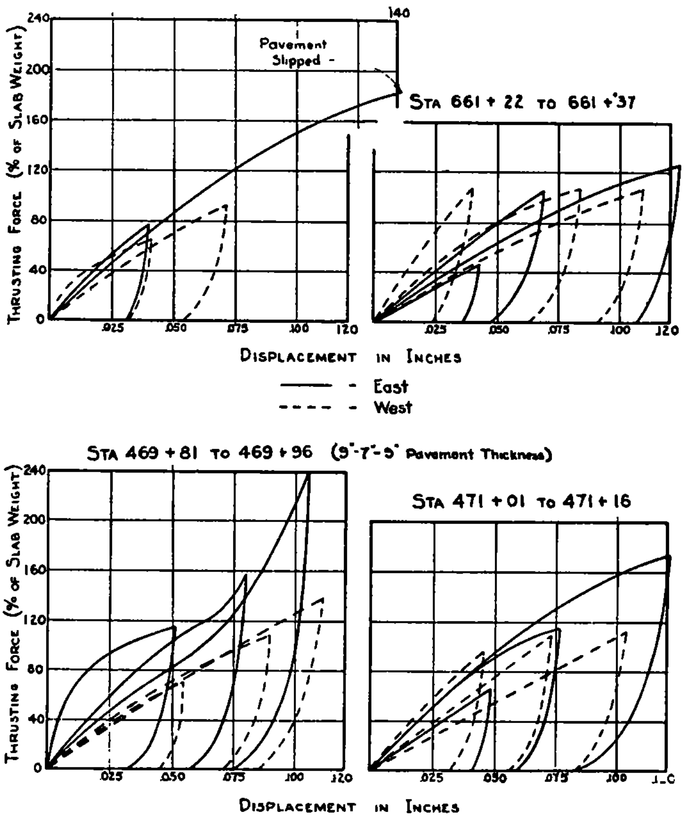


Figure 10. Force Displacement Curves, 8 in. Uniform Pavement Thickness

Thrusting force to move the slab was supplied by compressed air through a 4-in. soft suction hose placed in the 4-in. opening at the ends of the slab and attached to a controlled variable pressure source. Wood shims were used to reduce the width of the opening to within the calibrated range of the hose.

The slab was first moved east slightly more than 0.1 in. by applying pressure through the hose at the west end of the slab It was then moved west approximately 0.2 in. by applying pressure through the hose at the east end of the slab. The displacement was

that might be expected under a pavement after a number of movements resulting from moisture and temperature cycles.

At the conclusion of this preliminary treatment the slab was then displaced east 0.05 in. from the original position so that any subsequent movements would lie well within the range of the preliminary movements. The subgrade friction test was then started from this point. An increment of pressure sufficient to displace the slab approximately 0.01 in. eastward was applied slowly and held constant until all displacement had ceased.

After recording the pressure and displacement an additional pressure increment was applied slowly for a total displacement of about 0.02 in. and held constant until all forward displacement had ceased. The pressure increments were repeated until a total displacement of approximately 0.05 in. was obtained. The pressure was then slowly decreased by the same units used to induce the forward displacement. The pressure at each decrement was held constant until all movement had ceased and the recovery measured.

Pressure was then applied to move the slab in the reverse direction and displacement increments read as described for the first direction. Complete pressure and displacement measurements were secured for successive slab displacements of approximately 0 05 in , 0.075 in. and 0.1 in. The increments of pressure were selected so that about five sets of intermediate readings were secured for each overall displacement in each direction. The resultant pressure-displacement curves are shown in Figure 10.

As will be noted the subgrade friction on the full-sized concrete slabs increased for increased increments of displacement. The average value of the thrusting force to move the 8-in. uniform thickness slab 0 025 in. was about 40 per cent of the slab weight.

The average value for a movement of 0 05 in. increased to about 80 per cent of slab weight and to about 110 per cent of the slab weight for a movement of 0.1 in.

The average thrusting force for the displacement of the 9- 7- 9-in. slab 0.025 in. was about 45 per cent. The average value for a movement of 0.05 in. increased to about 90 per cent of slab weight and to about 130 per cent of slab weight for a movement of 0.1 in.

DETERMINATIONS OF k (WESTERGAARD)

Measurements were made by four different methods to secure information concerning Westergaard's modulus of subgrade reaction k. (1) A *diaphragm* method, (2) a *plunger* method made on the subgrade after it had been prepared for the paving operations were erratic and unreliable and are not further described in this report. (3) Full pavement thickness *concrete discs* of 12-, 18-, 24-, and 36-in. diameter were constructed on the subgrade during paving operations and (4) deflection measurements were made at the centers of full sized concrete pavement slabs (11 by 15 ft.).

Concrete Discs

Design. Circular forms made of 16-gage galvanized iron 5½-in. high and 12-,18-,24-, and 36-in. inside diameter were placed on the subgrade just prior to paving operations. The forms consisted of two concentric rings, the inner ring having the required diameter of the concrete disc, and the outer ring being 4 in. greater in diameter, thus leaving a 2-in. clear space between the rings. A spider was placed in the inner ring to hold the form to a circular shape while a bituminous plant-mix was firmly tamped in the space between the two rings. The forms were placed on the subgrade with the top 1½ in. below header grade. They were sufficiently rigid to maintain their shape while concrete was placed by hand inside and around the forms just ahead of the concrete placing operations and allowed normal pavement spreading and finishing operations to proceed without interruption. The concrete ring above the plant-mix was removed and the edges finished with a trowel after the concrete had hardened to the finishing stage.

Twelve discs, three of each size, were cast in the center of alternate 15-ft. slabs. See Figure 11.

Test Procedure: The load was applied to the 12-in. diameter concrete disc by means of a hydraulic jack placed between the frame of a loaded truck and a 12-in. diameter by ⅞-in. thick steel plate resting on the concrete (Fig. 12) This steel plate was set in a thin plaster of Paris paste to insure full bearing between the plate and the concrete surface. Penetration was measured at the quarter points with four Ames dials attached to arms spaced equal distances from the center and resting on the adjacent concrete slab.

The applied load was held for 5 min. The dials were then read and the load released for 5 min. after which the dials were re-read to determine the elastic rebound. The loading cycles were repeated until the difference in the dial readings between the load and no load conditions remained the same for at least two loading cycles.

The load was then increased and held for 5 min , the dials read and the load again released for 5 min. The load cycles were again repeated as for the first load increment until the difference in the dial readings between the load and no load conditions remained the same for at least two loading cycles.

The 18-in. diameter concrete discs were first capped with $\frac{7}{8}$-in. thick 18-in. diameter steel plates as described for the 12-in. diameter

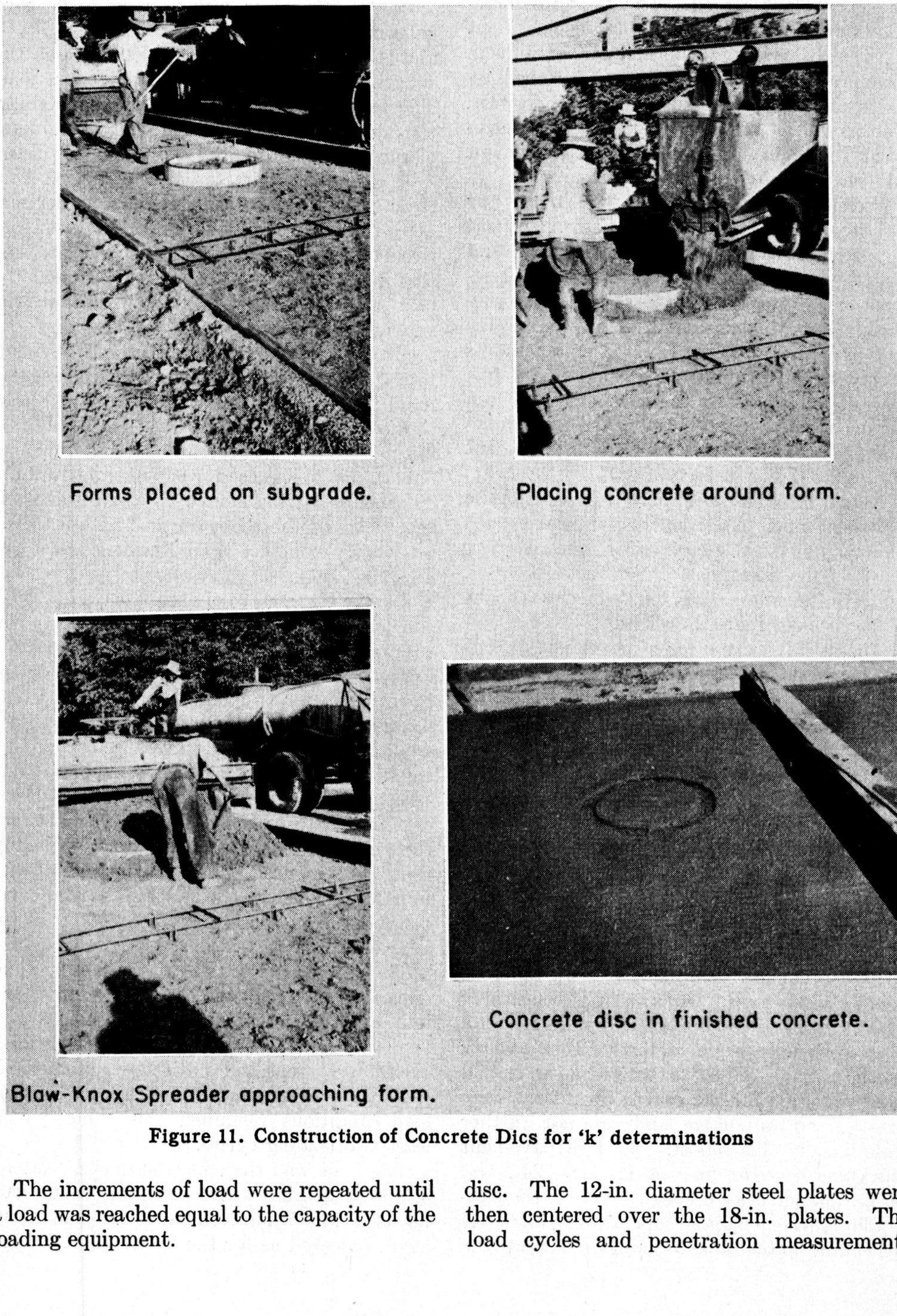


Figure 11. Construction of Concrete Dics for 'k' determinations

The increments of load were repeated until a load was reached equal to the capacity of the loading equipment.

disc. The 12-in. diameter steel plates were then centered over the 18-in. plates. The load cycles and penetration measurements were then made in the same manner as for the 12-in. diameter disc.

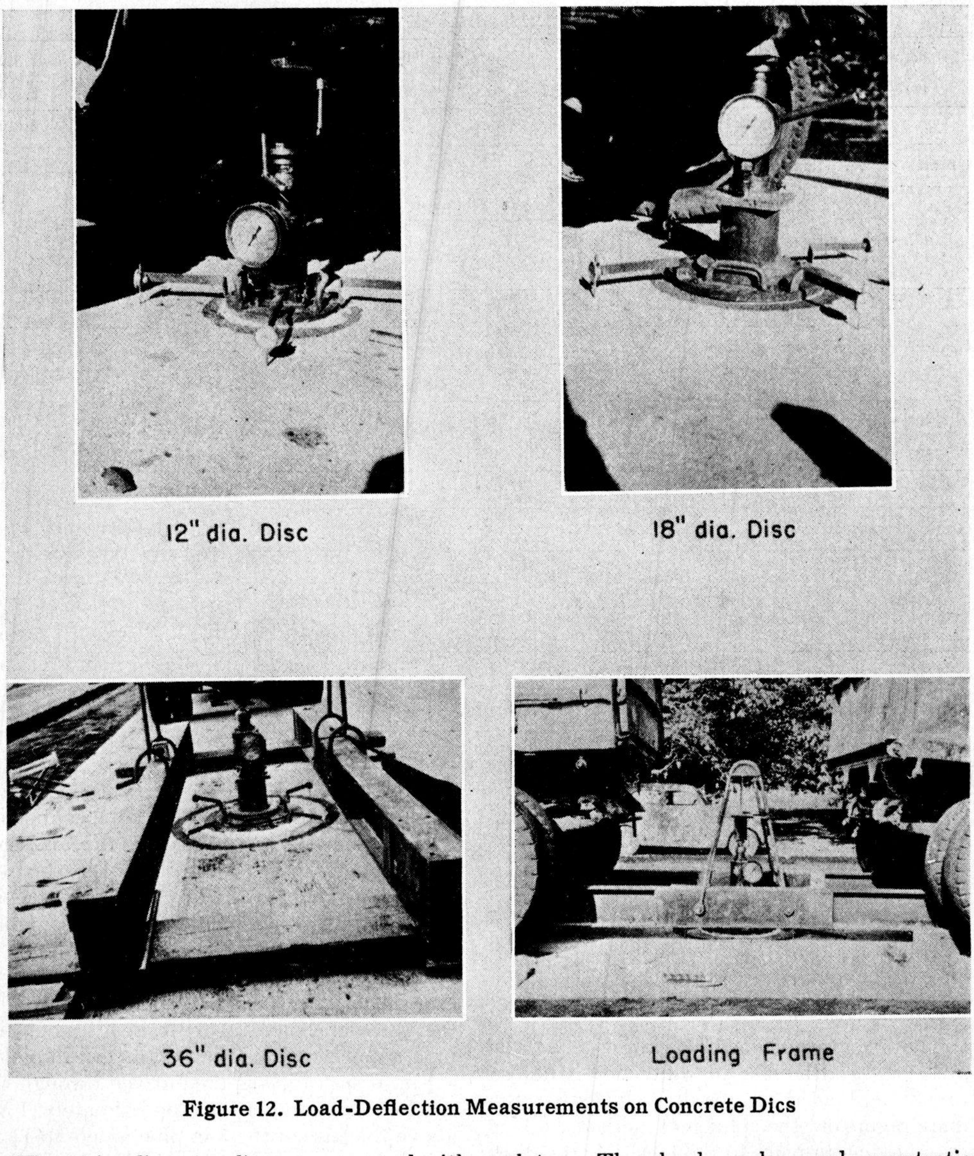


Figure 12. Load-Deflection Measurements on Concrete Dics

The 24-in. diameter discs were capped with a pyramid of $\frac{7}{8}$-in. thick 24-, 18- and 12-in. diameter steel plates. The load cycles and penetration measurements were then made in the same manner as described for the 12-in. diameter disc.

The same steel plate set-up was used for the tests on the 36-in. diameter concrete discs as on the 24-in. discs. This left a 6-in ring of concrete on the outside of the pyramid of steel plates. The load cycles and penetration measurements were then made in the manner described for the 12-in. diameter disc.

The averages of the $k(1)$[3] and $k(2)$[4] values

[3] k (1) slope of load determination curve for first application of load.

[4] k (2) slope of load determination curve for final application of load.

TABLE 8
SUBGRADE *k* (WESTERGAARD) DETERMINED FROM DEFORMATION OF CONCRETE DISCS

Sta	Load psi	*k*(1)	*k*(2)	Load psi	*k*(1)	*k*(2)	Load psi	*k*(1)	*k*(2)
12-in Diameter									
406 + 81				40.4	1,670	2,380	84.2	1,340	2,550
408 + 01	8.6	2,150	1,720	36.0	3,270	3,600	79.8	2,960	2,960
409 + 21	12.6	1,400	1,800	40.0	2,220	2,500	83.7	1,860	2,330
Aver	10.6	1,780	1,760	38.8	2,390	2,830	82.6	2,050	2,610
Grand Average, all loads									*k*(1) = 2,080, *k*(2) = 2,470
18-in Diameter									
407 + 11	16.9	1,540	1,880	36.3	1,730	1,910			
408 + 31	16.9	1,410	1,880	36.3	1,730	2,130			
409 + 51	18.1	1,390	1,650	37.5	1,630	1,700			
Aver	17.3	1,450	1,600	36.7	1,700	1,910			
Grand Average, all loads									*k*(1) = 1,580, *k*(2) = 1,860
24-in Diameter									
407 + 41	20.2	1,440	1,440	42.0	1,450	1,620	64.1	1,250	1,390
408 + 61	20.5	1,280	1,470	42.4	1,210	1,460	64.6	1,220	1,340
409 + 81	20.8	1,040	1,300	42.7	1,090	1,220	64.8	670	1,080
Aver.	20.5	1,250	1,400	42.4	1,250	1,430	64.5	1,050	1,270
Grand Average, all loads									*k*(1) = 1,190, *k*(2) = 1,370
36-in Diameter									
407 + 71	13.4	790	960	23.2	750	930	33.0	790	870
408 + 91	13.6	720	850	23.4	750	870	33.2	750	790
410 + 11	13.7	980	860	23.5	780	870	33.3	720	790
Aver	13.6	830	890	23.4	760	890	33.2	750	820
Grand Average, all loads									*k*(1) = 780, *k*(2) = 870

k(1) = Slope of load deformation curve for first application of load
k(1) = Slope of load deformation curve for last application of load

for all the loads for each diameter (from Table 8) were as follows:

	12-in. dia	18-in dia	24-in dia	36-in dia
k(1)	2,080	1,580	1,190	780
k(2)	2,470	1,860	1,370	870

The *k*(1) are somewhat lower than the *k*(2) values although the difference is not so pronounced as those obtained by either the diaphragm or the plunger methods. This may be due, in part, to the fact that the discs were cast directly on the subgrade while the other methods required a sand leveling course.

The values varied approximately as the square root of the area involved in the test.

Deflection of Full Size Slabs

Three-inch pipe couplings, fitted with a 3-in. nipple, were grouted into 6-in. core holes cut in the centers of the 15-ft. pavement slabs. The couplings were placed in the alternate slabs between those containing the concrete discs. A hole extending 5½ ft below the top of the pavement was made with a soil sampler through the center of the pipe coupling. A 5-ft. length of 1-in. thin wall conduit, to serve as a casing, was then pushed to the bottom of the sampler hole. A 6½-ft. length of ½-in. pipe, topped with a ½-in. by ¾-in. bushing and a ¾-in. brass cap, was then driven through the conduit extending from approximately 1 in. above the base of the 3-in pipe nipple to 1½ ft. below the casing. A ¾-in. brass cap was then drilled and tapped for a ¼-in. thread

The center of a 3-in. countersunk pipe cap was drilled and tapped for a standard ⅛-in. pipe thread. A collar threaded for ⅛-in. pipe thread was soldered to the case of a 0.0001-in. Ames dial. The dial was attached to the pipe cap with a 4-in. length of ⅛-in. standard pipe. A short length of ¼-in. threaded rod was screwed into the ¾-in. cap on the reference rod and extending inside the ⅛-in. pipe contacting the stem on the dial. The dial attached to the pavement through the 3-in. contact with the pavement so that it would not shift when a load was applied through a hydraulic jack to the pavement slab under test.

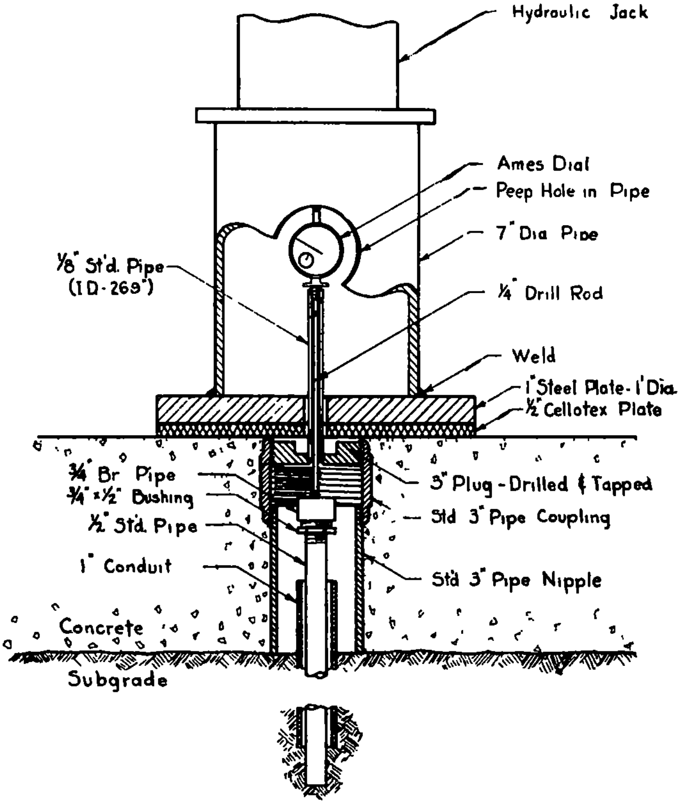


Figure 13. Schematic Diagram of Apparatus for Measuring Pavement Deflection

coupling measured the vertical movement of the pavement with reference to the rod anchored 5½ ft. below the surface.

The load was applied to the concrete adjacent to the couplings through a 12-in. diameter steel plate cushioned on the concrete with a ½-in. thick celotex gasket. The dial was read through a peephole cut in a piece of 7-in. pipe welded to the steel plate for supporting the hydraulic jack.

Figure 13 is a diagram of the apparatus.

An I-beam frame was placed on the slab under test, and a water tank backed up to each end, with the rear wheels resting on the adjacent slabs. Jacks were placed between each corner of the I-beam frame and the frame on the tank trucks, with sufficient pressure applied to the jacks to just keep the frame in

TABLE 9
SUBGRADE k (WESTERGAARD)
Center Loading—Full Size Slab

Station	Total Load	k for [a]E_s = 3 5 × 10^6	Total Load	k for E_s = 3 5 × 10^6
408 + 16	7,900	2,640	10,800	2,270
408 + 76	7,900	1,310	10,800	1,230
409 + 36	7,900	1,650	10,800	1,730
409 + 96	7,900	1,370	10,800	1,370
Average		1,750		1,650

$k(1) = k(2)$ for full size slab.
[a] E_s = The Assumed secant E at 28 days

Values of k (Table 9) based on the measured load deflection characteristics of a full-sized concrete slab were obtained by substituting in Westergaard's formula for Case II loading. Poisson's ratio was taken at 0.16. The thick-

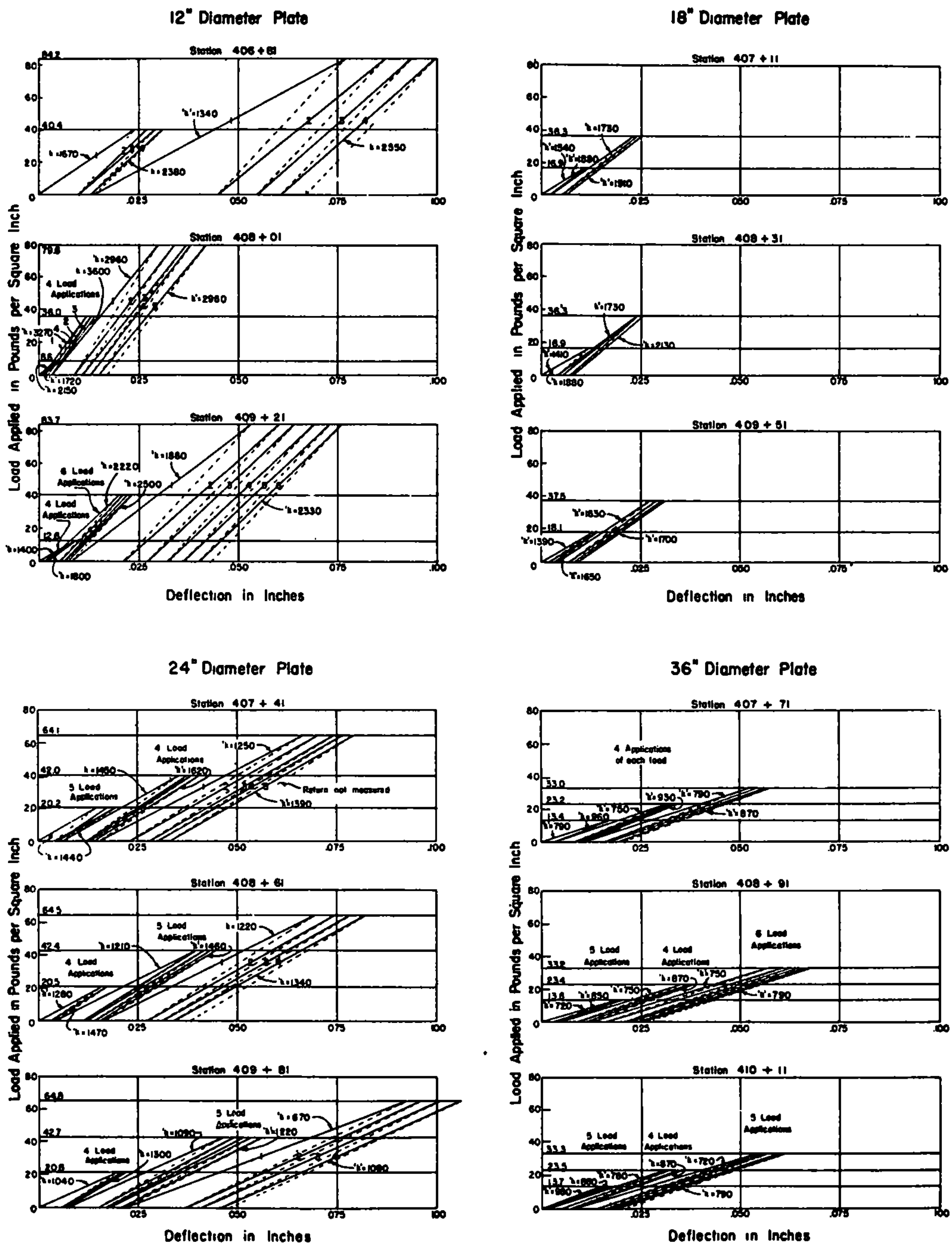


Figure 14. Load-Deflection Curves, K calculated from slope of lines

ness of the slab was 7-in. and the compressive secant modulus of elasticity at 28 days 3,500,000 at 1000 psi. Load-deflection curves are shown on Figure 14.

PAVEMENT ROUGHNESS MEASUREMENTS WITH THE PROFILOGRAPH

An instrument, termed a "Profilograph" (Fig. 15) designed by Senior Physical Testing Engineer F. N. Hveem and built in the materials and Research Department machine Shop, was used to measure changes in profile and roughness of the pavement at different times of the day and at different periods.

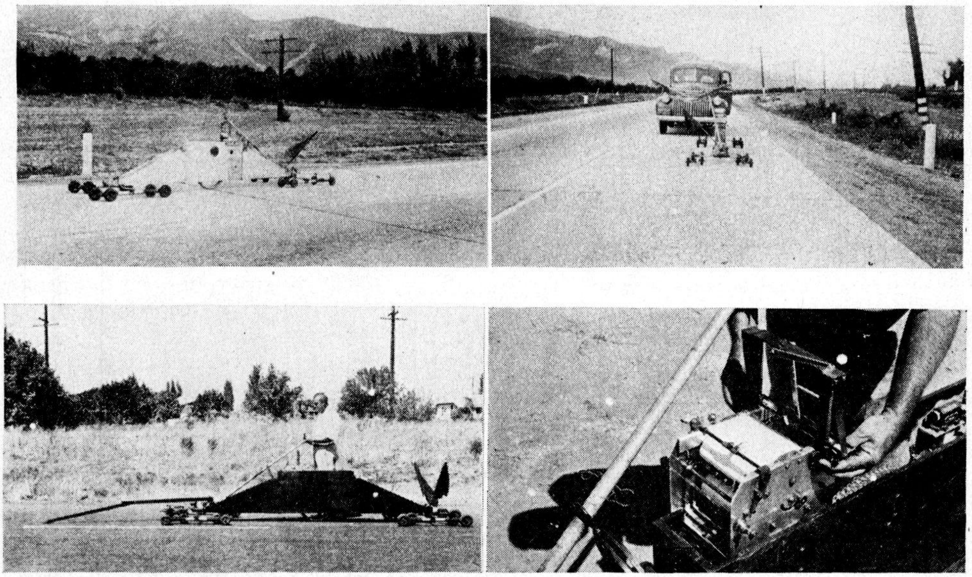


Figure 15. (Upper) Profilograph. (Lower left) Profilograph in Operation. (Lower right) Close up of Recording Device on Profilograph. (Pens not in place.)

The "Profilograph" is supported on 16 small wheels, arranged in a staggered pattern so that the body of the machine moves, as nearly as possible, in an even plane which corresponds to the general plane of the pavement surface.

In the center of the frame is a standard bicycle wheel so attached that it is free to move vertically and follow the contour of the pavement. Through a series of levers, etc., the movements of the bicycle wheel are transmitted to and recorded on a continuous graph. The respective horizontal and vertical scales on the graph are such as to make the actual contour of the pavement surface readily apparent.

By this instrument, seasonal or daily changes in surface contour may be observed, recorded and compared. Typical examples of graphs for portions of the Ventura experimental pavement are shown in Figures 16 and 17. These graphs illustrate very clearly the tendency of concrete slabs to curl upward at the joints under certain conditions of temperature and moisture content.

Such Profilograph records afford an excellent visual demonstration of the features which affect the riding qualities of concrete surfaces as influenced by the temperature and moisture as well as by construction deficiencies.

Complete Profilograph records were made on all test sections but are not included with this report because of space requirements. The records are available, however, for comparison at future measurement periods.

ACKNOWLEDGMENT

The following members of the staff of the California Division of Highways and their

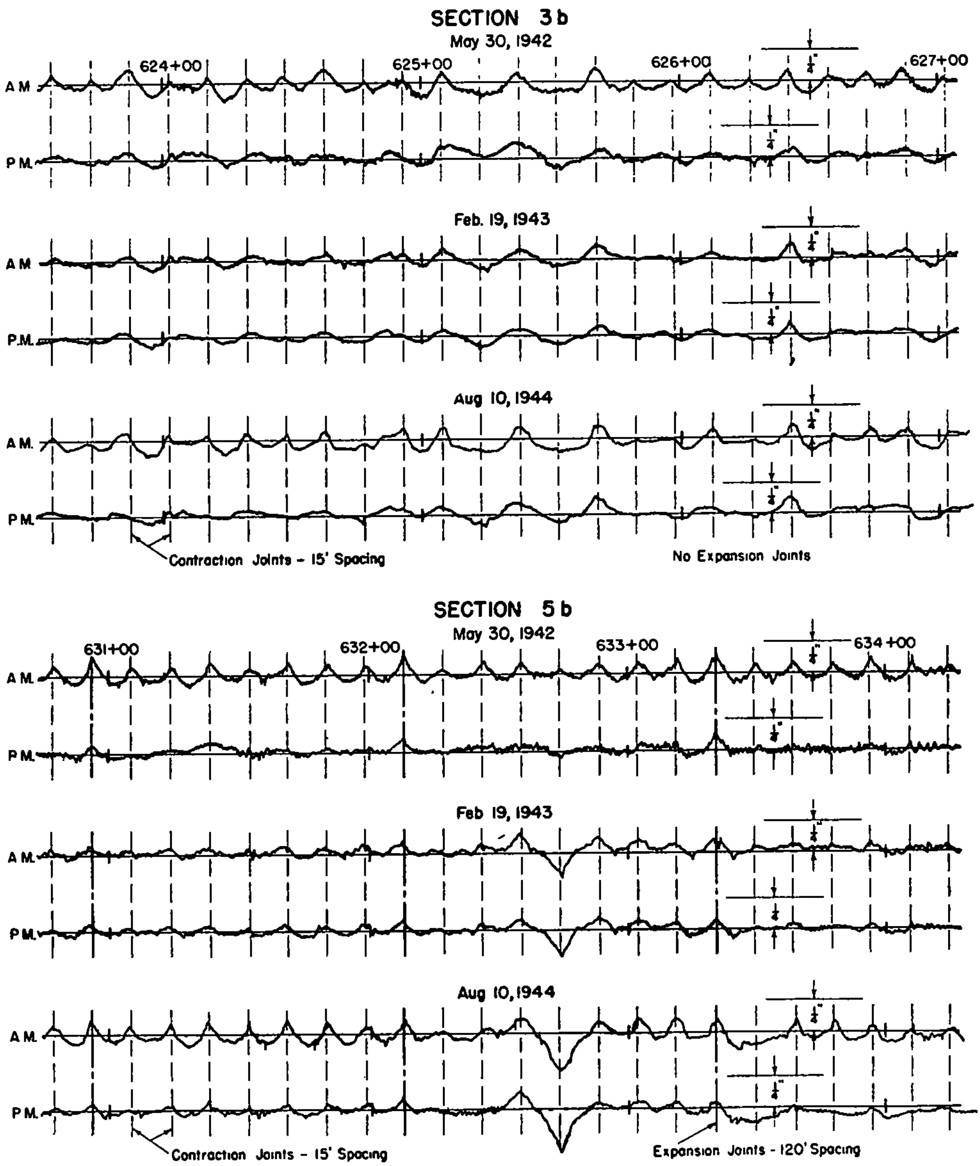


Figure 16. Ventura County Experimental Concrete Pavement; Comparative Profilograph Records

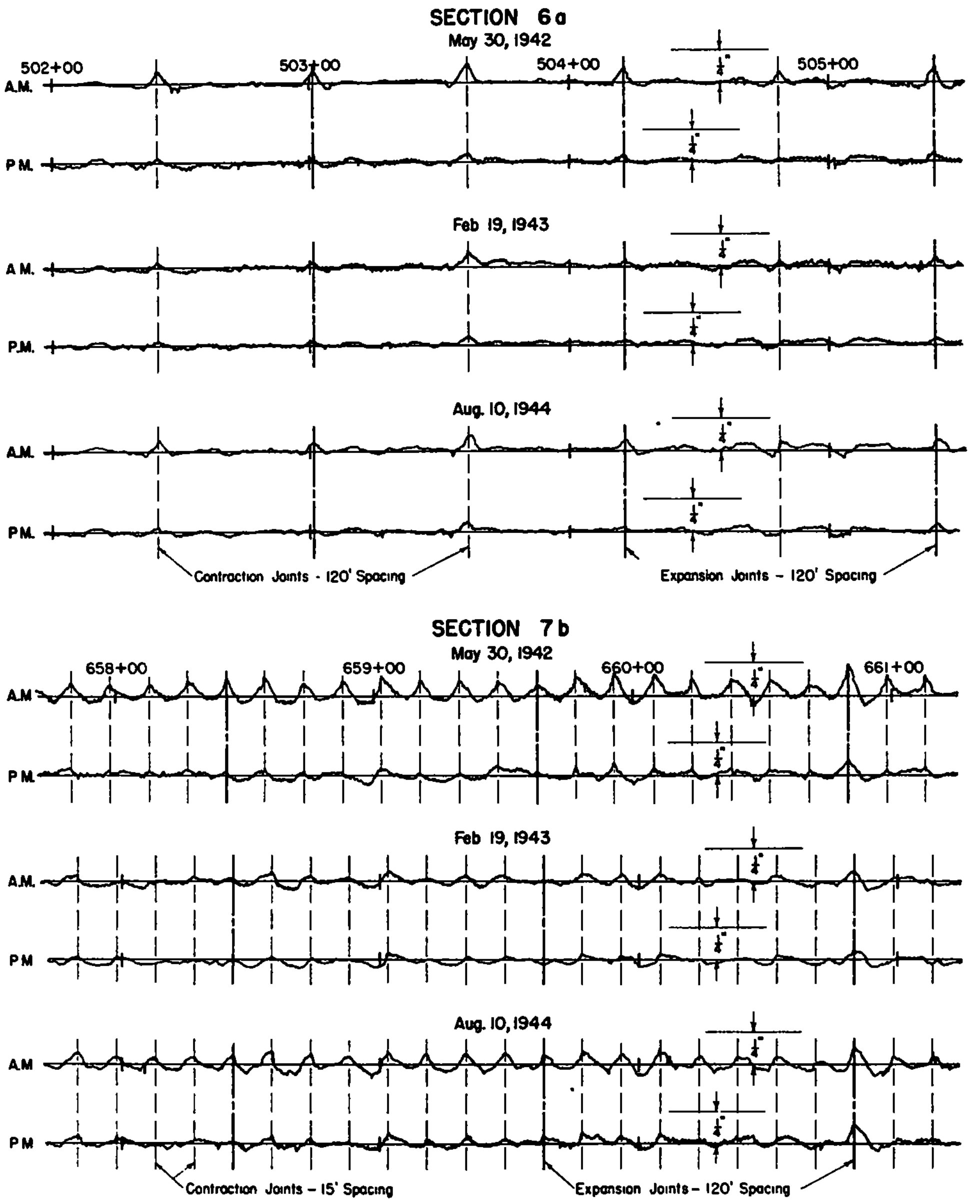


Figure 17. Ventura County Experimental Concrete Pavement; Comparative Profilograph Records

assistants actively participated in this work in the field

District Engineer Cortelyou, District Construction Engineer A N George, District Materials Engineer Rex Allan, Resident Engineer E. L Seitz

From headquarters at Sacramento, Assistant State Highway Engineer F J. Grumm, Construction Engineer R M Gillis and their assistants, E W Withycombe, A. M Nash and N. C McCorkle

The Materials and Research Department was represented by Senior Physical Testing Engineers, O. J Porter and F N Hveem and their assistants J. E. Barton and J L. Beatty.

The immediate direction of the investigational features of this project was the responsibility of the Materials and Research Engineer.

Messrs Barton and Beatty were primarily responsible for securing all field data relating to the concrete pavement and compiling and analyzing the data at Sacramento.

All work was done under the general direction of Director of Public Works C H Purcell and State Highway Engineer G. T McCoy and in cooperation with the representatives of the Public Roads Administration

INVESTIGATIONAL CONCRETE PAVEMENT IN KENTUCKY

By Thomas R Thomas, *Materials Engineer, Kentucky Department of Highways*

A description of the project including the construction data has been published in the 1940 Proceedings of the Highway Research Board.[1] This report covers the period from September 1940 through July 1944 and summarizes the observations and measurements made during that time.

Details of the seven different experimental sections are given in Table 1.

Expansion joints were constructed to accommodate a 1-in. width of premoulded bituminous fiber filler, and contraction joints are of the weakened plane type with a premoulded bituminous fiber filler

Dowel bars for load transfer were secured in proper spacing and alignment by welded dowel spacers which remained in place.

In sections where wire mesh reinforcing was installed the initial pour of concrete was struck off 2 in below grade for placing of the mesh.

[1] F. P Anderson, 2nd, "Investigational Concrete Pavement in Kentucky," *Proceedings*, Highway Research Board, Vol. 20, p 337 (1940)

PHYSICAL PROPERTIES OF THE CONCRETE

Daily job control specimens were made and broken at 28 days. The average cylinder

TABLE 1
DESIGN OF EXPERIMENTAL SECTIONS

Section No	Length	Design Section	Wire Mesh Reinf	Expansion Joints		Contraction Joints	
				Spacing	Load Transfer	Spacing	Load Transfer
	ft	*in*		*ft*		*ft.*	
7	1,250	7-7-7	None	120	None	20	None
6	1,500	9-7-9	70 lb	60-alt	Dowels	60-alt	Dowels
5	1,500	9-7-9	None	120	Dowels	20	Dowels
4	1,500	9-7-9	None	120	Dowels	20	None
3	2,500	9-7-9	None	400	Dowels	20	None
2	3,000	9-7-9	None	800	Dowels	20	None
1	5,000	9-7-9	None	None	None	20	None
Std [a]	7,000	9-7-9	44 lb	120	Dowels	30	Dowels
2-R	2,500	9-7-9	None	800	Dowels	20	None
3-R	2,500	9-7-9	None	400	Dowels	20	None
4-R	1,500	9-7-9	None	120	Dowels	20	None
5-R	1,500	9-7-9	None	120	Dowels	20	Dowels
6-R	1,500	9-7-9	70 lb	60-alt	Dowels	60-alt	Dowels
7-R	1,200	7-7-7	None	120	None	20	None

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