Colorado Concrete Pavement and Subbase Experimental Project

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In 1952 the first two lanes of a planned four-land concrete highway between Denver and Castle Rock, Colo. were placed. The alignment for the highway traversed areas having soil of high swell characteristics. Within a short time after placement, the pavement warped and cracked in certain areas. In 1956 the additional two lanes were placed. In this construction, the subgrade was adjusted on the wet side of optimum moisture for the specified compactive effort. Five pavement structures were incorporated in the job: an 8-in. plain concrete slab built on a 4-in. granular subbase treated with 2 percent portland cement, a 10-in. plain pavement on a 4-in. subbase, an 8-in. plain pavement on a 20-in. subbase, and two mesh-reinforced 8-in. pavements on a 4-in. subbase. The joint spacing was 106 ft in one reinforced section and 61 ft in the other.

During the spring of 1959 the five experimental designs were tested under dynamic and static wheel loads. Deflections and strains due to dynamic loads and deflections under static loads were measured. The modulus of subgrade reaction was determined from plate bearing tests.

Data from the tests show that the reinforced pavement designs having a relatively long joint spacing undergo greater corner deflections under load, and attain higher curl than the plain concrete pavement having relatively short joint spacing. An experimental section having a 20-in. cement-treated subbase gave lowest corner deflection.

Only limited conclusions are drawn pending further visual observations and testing after a longer period of service.

● THE GRADING of the first two lanes of a planned four-lane highway between Denver and Castle Rock, on US 87, was completed during the years 1950 and 1951. The alingment for this new highway traversed areas having soils of high-swell characteristics. Specifications for grading and compaction required a minimum density of 90 percent of AASHO modified density (1).

In 1952 pavement was placed on the completed subgrade. This consisted of a 6-in.

layer of granular subbase material and an 8-in. uniform thickness portland cement concrete pavement. Shortly after the pavement was placed, cracking and warping of slabs was observed in certain areas. It was generally believed that the amount of water used during grading operations was not sufficient to allow the swelling soils to reach equilibrium before the pavement was placed.

The grading work for the additional two lanes was placed under contract in 1956. Specifications for this grading required 95 percent of AASHO T 99 standard density (2). This project was constructed slightly on the wet side of optimum moisture for the specified compactive effort. It was believed that the swell potential of the swelling soils had thereby been largely reduced. A pavement design was chosen as shown in Figure 1. The granular subbase material (4 in. thick) was treated with 2 percent portland cement to produce a minimum stabilometer resistance value of 80, not a hardened soil-cement material. An 8-in. uniform thickness portland cement concrete pavement was considered adequate. This design is referred to later as the "typical section."

It was realized that this pavement project afforded an excellent opportunity to compare the performance of similar subgrade soils compacted under AASHO T180 modified compaction and under AASHO T99 standard compaction. At the same time, it was considered advisable to include some variables in pavement design. If the new pavement should remain smooth, it would be assumed that sufficient moisture had been added to prevent swelling of the subgrade soils and the pavement variables might not provide any pertinent information. On the other hand, if distortion and cracking should occur, the pavement variables might provide some information regarding their relative merits. Later, it developed that there would be an opportunity to load-test the various pavement sections.

PAVEMENT DESIGN VARIABLES

Five experimental pavement-subbase designs were included in the project, as follows:

Section A (Station 377+49 to 403+96)—2,647 linear feet of 8-in. concrete pavement reinforced with 6 x 12¼ welded wire fabric (weighing 61 lb per 100 sq ft), on 4-in. cement-treated subbase. Fabric placed 2 in. below pavement surface. Sawn contraction joints spaced at 61 ft, 6-in. centers. Dowels: 1-in. round, spaced at 12-in. centers.

Section B (Station 403+96 to 429+99)—2,603 linear ft of 8-in. concrete pavement, reinforced with 6 x 12⁰⁰/4 welded wire fabric (weighing 79 lb per 100 sq ft), on a 4-in. cement-treated subbase. Fabric placed 2 in. below surface of pavement. Sawn contraction joints spaced at 106 ft, 6-in. centers. Dowels: 1-in. round, spaced at 12-in. centers.

Section C (Station 573+00 to 625+83.6)—5,283.6 linear ft of "typical section": 8-in. plain concrete pavement on 4-in. cement-treated subbase, sawn contraction joints at 20-ft centers. No dowels.

Section D (Station 625+83.6 to 652+23.6)—2,640 linear ft of 10-in. plain concrete

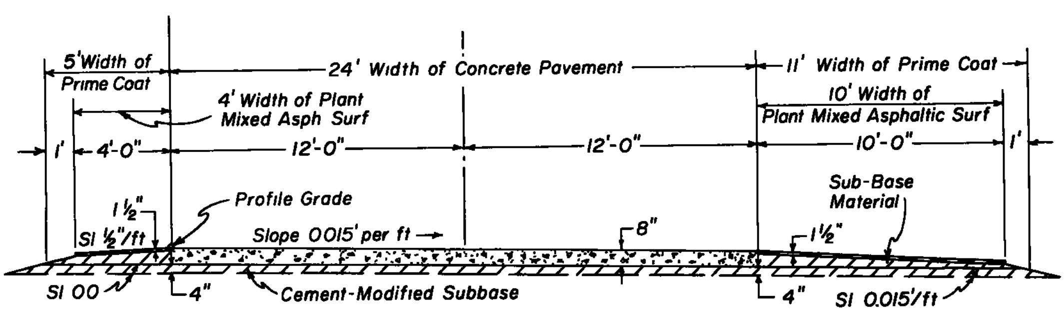

Figure 1. Typical section.

pavement on 4-in. cement-treated subbase. Sawn contraction joints spaced at 20-ft centers. No dowels.

Section E (Station 656+00 to 680+00–2,400 linear ft of 8-in. plain concrete pavement on 20-in. cement-treated subbase. Sawn contraction joints spaced at 20-ft centers. No dowels.

MATERIALS AND CONSTRUCTION

The predominant subgrade soils in the top 3 ft of all five experimental pavement sections are within the range of A-7-6 (12) to A-7-6 (20).

A typical grading of untreated material for the cement-treated subbase is as follows:

Passing 3/8 in.	100%
Passing No. 4	99%
Passing No. 10	90%
Passing No. 40	52%
Passing No. 100	26%
Passing No. 200	22%
Plasticity Index	8

After treatment with 2 percent portland cement, Atterberg tests showed the subbase material to be non-plastic.

Aggregate for the 8-in. concrete pavement had a maximum size of 2 in. The cement factor was 6 sk per cu yd and the average 28-day compressive strength of test cylinders was 4,244 psi. Entrained air in the fresh concrete was maintained between 4 and 5 percent.

The pavement was placed by means of a slipform paver (3). This necessitated the use of concrete having a slump of about 1¾ in. to 2 in. For placement of the mesh reinforcement, an improvised frame was attached to the front of the slipform paver. As the frame was pushed ahead of the paver, it held the wire mesh off the prepared subbase at the required height of 6 in. (2 in. below the finished surface of the pavement).

Although no traffic count has been made at the exact location of the test sections, counts made in 1958 (year of completion of pavement) at junctions about 10 mi north and south, indicate the average daily vehicle (ADV) count to be 6,000 for the 4-lane facility. However, the completion of a connecting freeway through Denver may have increased the ADV by as much as 20 percent.

After completion of paving operations, profile elevations were taken on all the test sections. Elevations were taken again approximately 1 yr later. Although some uplift has been observed, trends are not sufficiently developed to warrant a conclusion at this time. Pavement distortions that have occurred on the five test sections are not nearly as severe or widespread as those which have occurred on the opposite two lanes placed under drier soil conditions in 1952.

As an additional check on pavement distortion, an as-completed profile of the pavement was made during the summer of 1958 using a California profilometer. It is expected that similar observations to be made in the future will trace the changes in pavement profile.

LOAD TESTING PROGRAM

The load testing program was carried out during the spring of 1959 as a cooperative project of the Colorado Department of Highways and the Portland Cement Association.

Static and moving load tests were made, and strains in the concrete as well as deflections of the pavement were measured. These data permitted a comparison of the 5 experimental pavement designs. The instrumentation installed for measuring deflection was semi-permanent, and it is anticipated that additional tests will be made in 2 or 3 years.

A 4-axle semitrailer and tractor combination (Fig. 2) was used for loading, and measurements of pavement performance were made by maximum-reading deflectome-

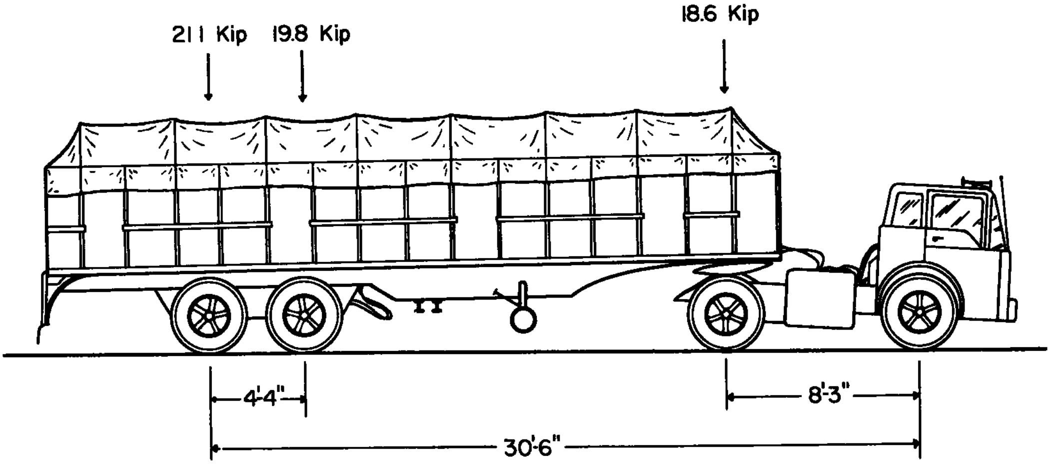

Figure 2.

ters and A-9 strain gages. Details of the loading and instrumentation are described later. Temperatures of the pavement surface, of the interface of the pavement and subbase, and of the air (shaded and unshaded) were registered on a continuous recorder.

Plate bearing tests were made on the subbase. Compression tests and static modulus of elasticity tests were made on 6-in. cores removed from each pavement section.

Three test sites were chosen from each of the experimental sections A through E. Each test site included four successive slabs which were instrumented as shown in Figure 3. Moving loads were applied to all of the sites by moving the loading truck in two main loops, each approximately 6 mi long. One loop included the test sites

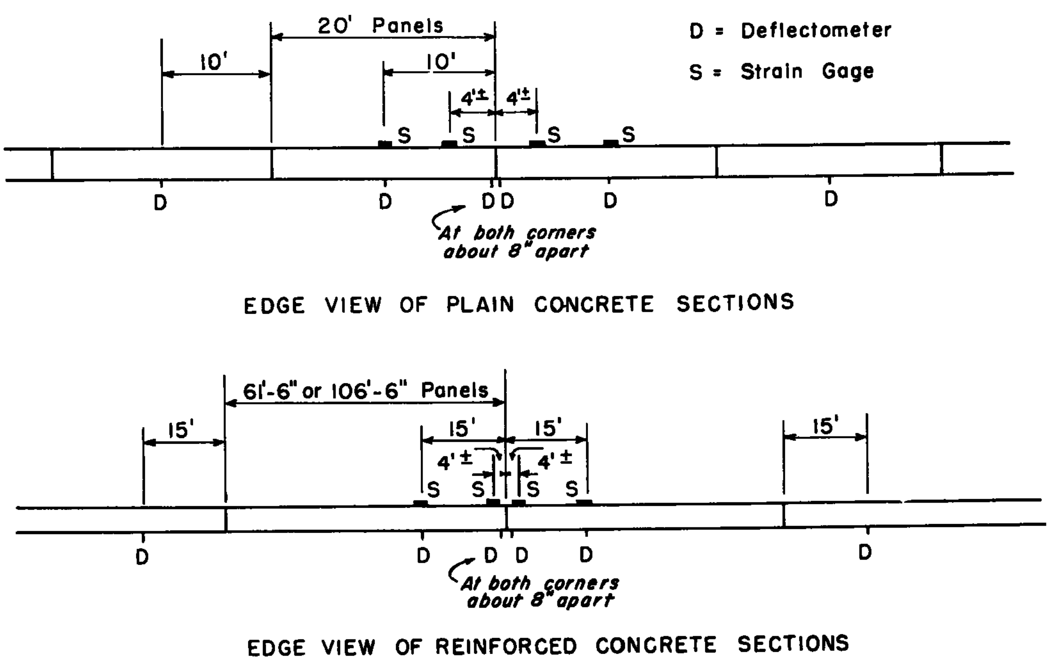

Figure 3. Location of deflectometers and strain gages

in sections A and B; the other included the test sites in sections C, D, and E. Loading tests were made three times during the day: between 5:30 a.m. and 7:00 a.m., between 10:30 a.m. and 12:00 noon, and between 3:00 p.m. and 4:30 p.m. Each loading test consisted of at least 3 trips of the truck over the instrumented slabs.

INSTRUMENTATION AND TEST PROCEDURE

The semitrailer truck was loaded with sand ballast so that the single-axle load was 18,600 lb and the tandem-axle load was 40,900 lb, of which 19,800 lb was on the forward tendem axle and 21,100 lb on the rear tandem axle (Fig. 2). Tire pressures were maintained at 80 psi. All load tests were made with the wheels of the truck running within 2 to 3 in. of the edge of the pavement at 2 to 3 mph. Outside wheel loads due to regular traffic normally would be about 30 in. from the pavement edge. However, occasional loadings nearer the edge are inevitable and would represent more severe loading conditions. These tests, intended to form the basis of a design comparison, were made near the edge so that design differences would be maximized.

The type of maximum-reading deflectometer used is shown in Figure 4. The "sensing element" consists of a vertical steel pin held in a machined brass housing by a leather friction pad. The housing is attached to an adjustable bracket which is in turn attached to the vertical edge of the concrete slab in such a way the deflectometer pin rests on an 8-ft bench rod driven through a 5-ft long casing into the subgrade. During passage of the test load, the pin is displaced vertically relative to the housing. The displacement corresponding to maximum deflection of the pavement is retained, and is measured later by a 0.001-in. dial indicator equipped with an adapter. During

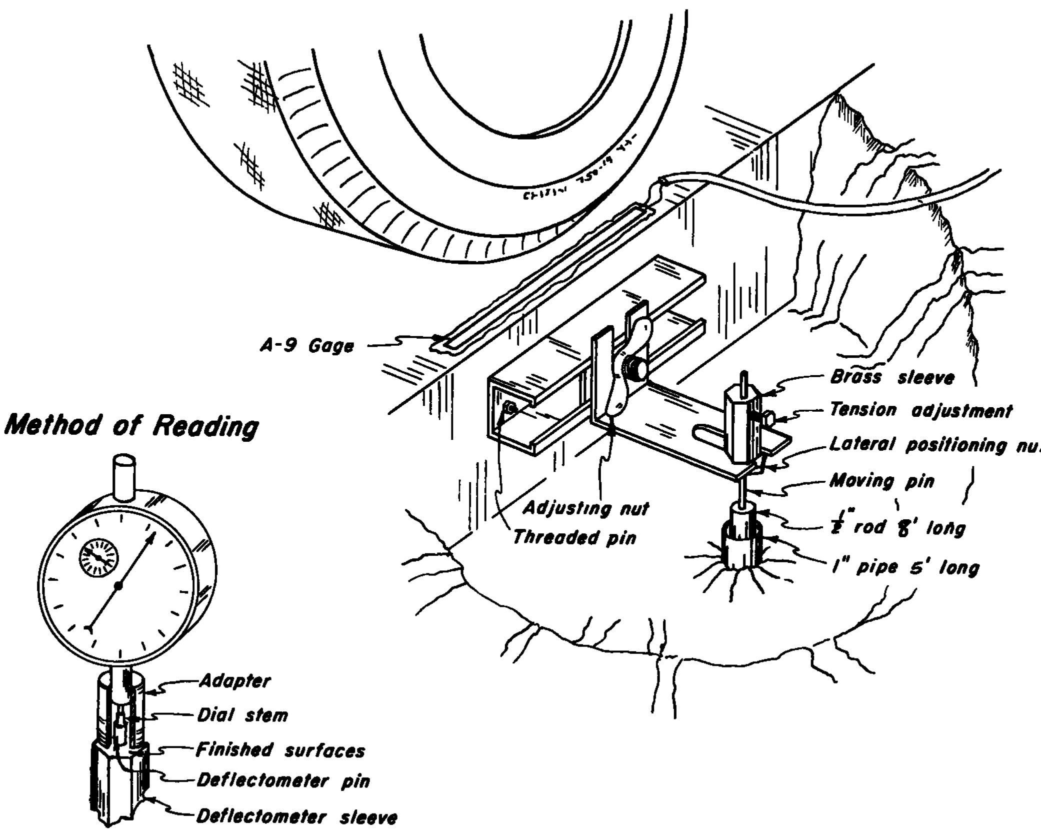

Figure 4. Deflectometer installation.

the day the change in pin position of the unloaded slab was also used for measuring the change in curl of the slab.

Strain gages were bonded to the surface of the pavement near the edge and were waterproofed with a tough wax. Changes in strain as the truck wheels approached and passed each instrumented point were recorded as traces on a strain indicator housed in an instrumentation trailer.

Plate bearing tests on the subbase were made by applying loads through a 30-in. steel plate 1 in. thick using the loading truck as a reaction for the hydraulic ram. For each test an excavation was made in the shoulder material to a level corresponding to the bottom of the pavement. The plate was placed on a thin layer of plaster of Paris and leveled. Four dial indicators were used to measure the downward motion of the plate. The dials were attached to wooden bridges supported at the pavement edge on one side and extending laterally to a point on the shoulder about 10 ft from the pavement edge.

FIELD TEST RESULTS

Temperature and Curl

The load-deflection characteristics, and to some extent the load-strain characteristics, of a concrete pavement are dependent on the extent to which the slab is curled upward or downward at the edges and the corners. Daily changes in curl are due principally to the differences in temperature between the top and bottom of the slab. Moisture content differences also contribute to curl but mostly on a "long-time" basis rather than a day-to-day basis (excluding the effects of rain).

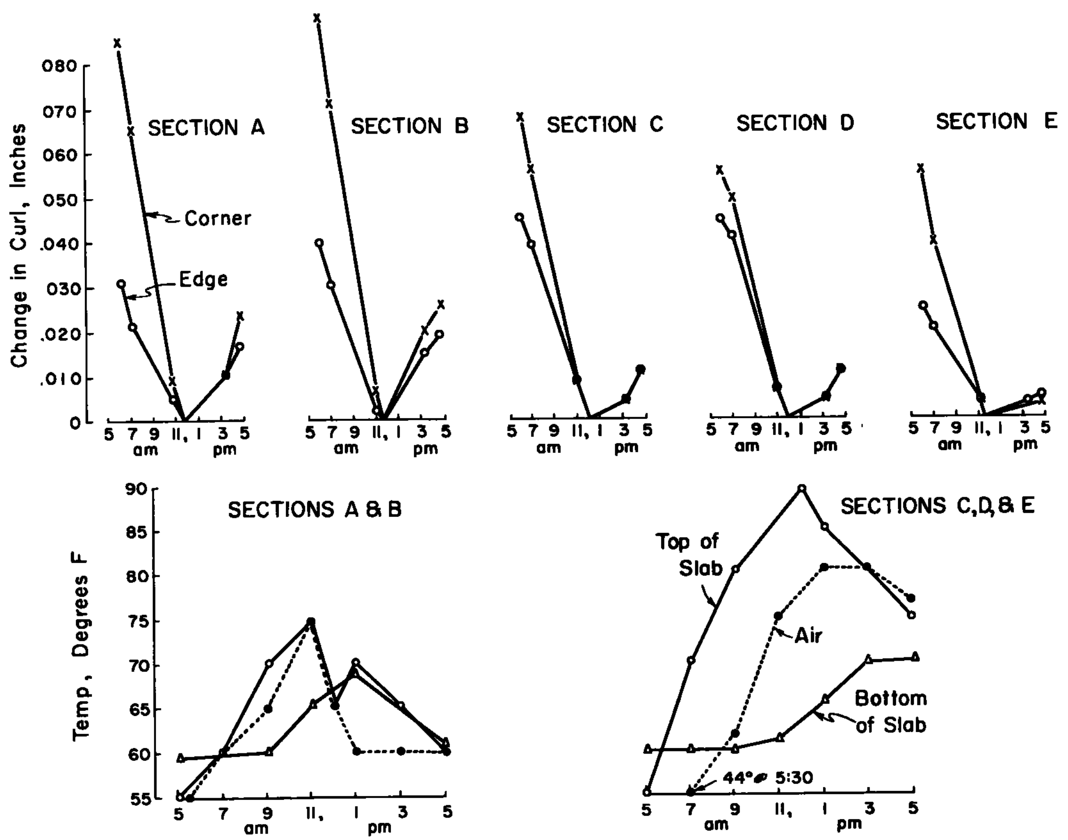

Figure 5. Temperature and curl vs time.

The changes in temperature of the surface of the slab and of the bottom of the slab for test installations in each of the 5 sections are shown in the lower part of Figure 5, and the changes in curl with respect to temperature are shown in the upper part of the figure. The plotted points for "change in curl" represent the average of 6 corner measurements and 12 edge measurements from each section. The change in curl was very rapid during the early morning hours when the temperature of the top of the slab was increasing rapidly. During the middle of the day all slabs ceased their downward curling and began to curl in the opposite direction.

The change in curl at the corners of sections A and B (long joint spacing) was greater than in the other sections tested even though a rapid drop in temperature of the top of the slabs occurred at 11:00 a.m. due to a brief rain shower.

Deflections–Moving Load Tests

In general, the deflection results reflected fairly consistent behavior of the test slabs in any one section. The data from at least 3 test runs over the slabs in each of the three sites of a section were averaged to permit a comparison of the structural behavior of the pavement in the 5 test sections. These deflection data are given in Table 1 for corner and edge positions. It will be recalled that these are maximum deflections registered by the passage of a loaded truck having a 40,900-lb tandem-axle load. The early morning corner deflections of reinforced sections A and B with the longer joint spacing are higher than those of the other sections. Corner deflections were lowest in section E regardless of time of day.

TABLE 1

AVERAGE OF MEASURED MOVING LOAD DEFLECTIONS (0.001 in.)

	Corner					Edge				
	Sections					Sections				
	A	B	C	D	E	A	B	C	D	E
Early a.m.	47	50	24	19	19	26	22	22	17	11
Late a.m.	17	11	8	11	7	11	10	9	11	11
Late p.m.	13	13	7	13	7	13	12	8	14	9

It may be seen from Figure 5 that the temperature conditions existing when sections A and B were tested differed from those existing during the testing of C, D, and E. Therefore, the data in Table 1 for the different experimental sections are not directly comparable. This is particularly true of the readings made in the early morning when it is known that the degree of upward curl of the pavement, and hence the deflections under load, can vary significantly on different days. Late morning and early afternoon readings may be compared with less question.

Deflections–Static Load Tests

Static wheel tests were made by placing each wheel of the truck at the edge of the slab at successive instrumented points. Measurements of deflection were made on each side of the joint for each wheel position. Table 2 gives measured deflections on either side of a joint when the load is on the approach slab, together with edge deflections. No great differences are apparent in corner deflections of sections A, C, and D. However, the deflections of section E, are significantly lower than those of the other sections and the deflections of section B appear to be significantly greater. This may be explained by the fact that section E has a 20-in. cement-treated subbase which may reduce deflections, and section B has the longest joint spacing which results in wider joint openings and more

TABLE 2

STATIC WHEEL LOAD DEFLECTIONS
(0.001 in.)

Time	Section A				Section B				Section C				Section D				Section E			
	Single		Tandem		Single		Tandem		Single		Tandem		Single		Tandem		Single		Tandem	
	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.	Ap.	Lv.
(a) Corner																				
a.m.	15	14	20	18	30	25	37	26	13	10	18	14	20	20	28	27	6	5	7	7
p.m.	12	12	18	16	14	14	20	18	8	8	10	8	14	15	23	22	5	5	7	7
(b) Edge																				
a.m.	10		16		13		21		9		17		19		31		6		10	
p.m.	11		17		9		16		7		13		12		25		9		11	

Ap. = Slab on which truck approaches joint (loaded slab).
Lv. = Slab on which truck leaves joint (unloaded slab).

curl which may reflect in higher corner deflections. It is unusual that the deflections of section D, the 10-in. thick pavement, are just as great as those of the 8-in. pavements.

The amount of load transfer attributed to aggregate interlock and to dowels is reflected by the difference in deflection of one side with respect to the other. Joint "effectiveness" was computed by the method of Teller and Sutherland (4), using

$$E_1 = \frac{2\,d_j'}{d_j + d_j'} \tag{1}$$

in which d_j' is the deflection of the unloaded joint-edge and d_j is the deflection of the loaded joint-edge. Values computed by using the expression are given in Table 3.

TABLE 3

JOINT EFFECTIVENESS
(Computed from Data in Table 2(a))

	Section A		Section B		Section C		Section D		Section E	
	S	T	S	T	S	T	S	T	S	T
a.m.	0.97	0.95	0.91	0.83	0.87	0.88	1.0	0.98	0.91	1.0
p.m.	1.0	0.94	1.0	0.95	1.0	0.89	1.0	0.98	1.0	1.0

S = effectiveness under single-axle loading.
T = effectiveness under tandem-axle loading.

The "joint-effectiveness" ratio is relatively high for all sections; however, sections D and E show some advantage over the other sections. It should be mentioned that the joint-effectiveness ratio will undoubtedly change during the year, and that in cold weather, when the slabs are in a contracted condition, aggregate interlock may be reduced in sections C, D, and E. This would reflect in a lower joint-effectiveness ratio.

Pavement Strains

Strains were measured along the edge of the slabs at two sites in each of the 5 sections. Gages were placed at 4 ft and 10 ft from the corner in the short slabs of sections C, D, and E, and at 4 ft and 15 ft in the longer slabs of A and B sections (Fig. 2). Averages of the strain readings for each section are given in Table 4. Strains were measured at creep speed and were recorded as a continuous trace (Fig. 6). Although both compressive and tensile strains were recorded, only the compressive strains in the top of the slab occurring as the wheels moved to a position adjacent to the gage are given in Table 4, because these were the larger strains. It has been

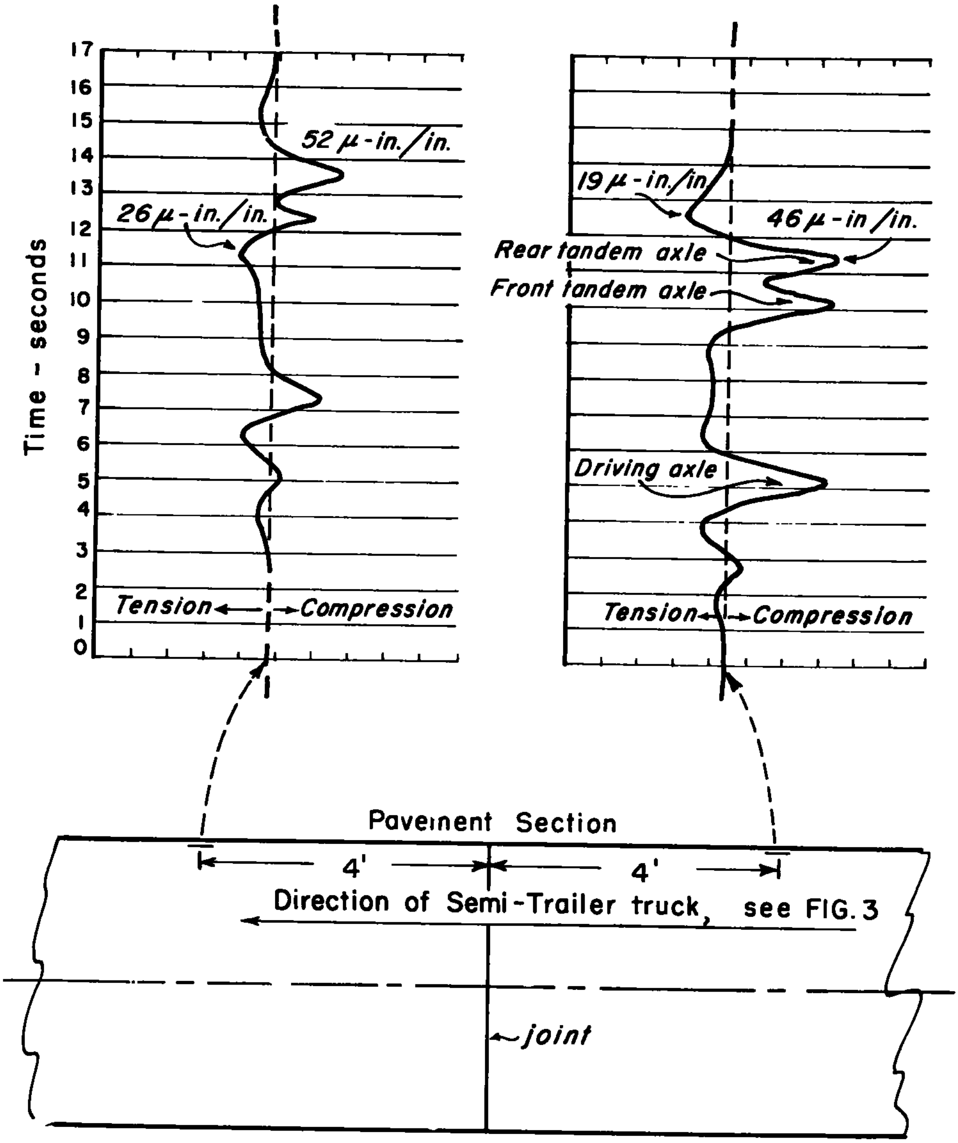

Figure 6. Strains near joint semi-trailer truck moving along slab edge.

TABLE 4

AVERAGE STRAINS
(Millionths)

		Section A			Section B			Section C			Section D			Section E		
		S	FT	RT	S	FT	RT	S	FT	RT	S	FT	RT	S	FT	RT
A.M., 4 ft from joint	AP	51	36	54	52	48	44	44	28	42	47	29	49	40	33	50
	LV	43	34	49	41	23	44	48	29	46	33	26	31	40	30	34
A.M., 10 or 15 ft from joint	AP	29	25	35	55	41	58	43	30	43	41	28	44	37	28	40
	LV	32	27	43	33	25	39	54	36	55	37	29	38	33	25	35
P.M., 4 ft from joint	AP	53	50	61				45	32	39	49	40	52	39	27	41
	LV	39	26	49				46	34	44	45	31	45	43	28	42
P.M., 10 or 15 ft from joint	AP	30	28	39				40	27	40	45	35	42	43	39	48
	LV	19	15	22				41	28	40	41	39	44	43	27	44

S = Single driving axle.
FT = Front axle of tandem.
RT = Rear axle of tandem.
AP = Slab on which truck approaches joint.
LV = Slab on which truck leaves joint.

anticipated that the tensile strains near a joint, when the load was at the joint, might be critical in some cases. However, as load transfer was good, tensile strains at this location were never very great. The strains measured in section A, 15 ft from a joint, were unusually low as compared with strains in other sections. No reason for this is readily apparent.

Strains of the order measured in these tests developed stresses ranging from 50 psi to 200 psi. Stresses in this magnitude should have no harmful structural effect on the pavement.

Supplementary Tests

Curves showing the load-deflection characteristics of the subbase-subgrade are shown in Figure 7. Values obtained at section C are believed to be influenced greatly by a local shale deposit near the surface at the test site. Results for sections B and E are in the order of those to be expected. The 20-in. layer of cement-modified soil in section E resulted in a very high k value of 600 pci when computed by a simple formula of unit load on the plate divided by the average plate deflection at that load.

Representative values for the compressive strength, the static modulus of elasticity in compression, and Poisson's ratio were obtained from compression testing a 6-in. core from each of sections A, C, D, and E. These data are given in Table 5.

CONCLUSIONS

It is believed that tests of the type described in this paper provide significant information as to the probable future behavior of a concrete pavement. Although only limited conclusions may be drawn from this one load-testing program, it is not unreasonable to believe that correlation of these tests with duplicate tests planned for 2 to 3 yr hence, plus interim visual observations, may provide a behavior pattern by which other roadways of this type may be evaluated early in their life.

Insofar as deflections and strains are concerned, all values were relatively low. Therefore, differences observed in the behavior of the various experimental sections are not considered significant at this time. The 8-in. plain concrete design of section C compared favorably with the 8-in. reinforced designs of sections A and B. Section

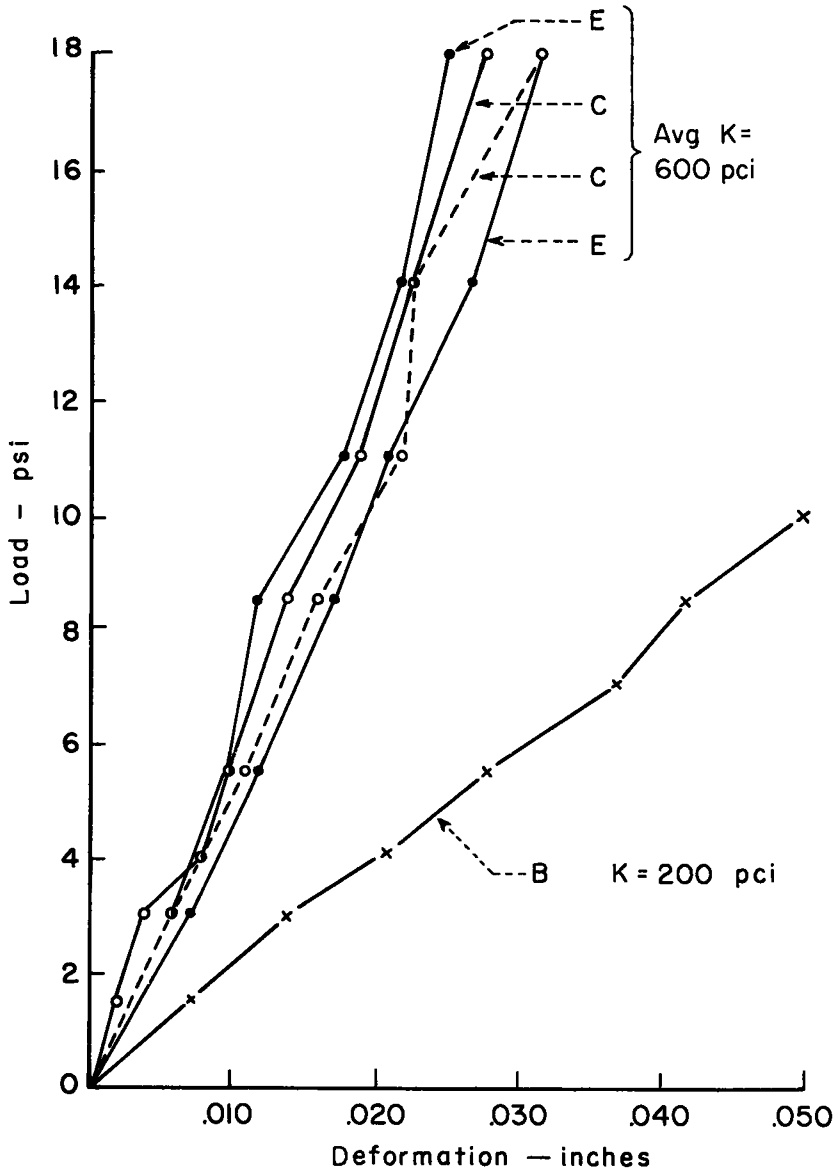

Figure 7. Plate bearing tests load vs deformation.

TABLE 5

COMPRESSION TEST RESULTS-6-IN. CORES TESTED DRY

Section	Compressive Strength, psi	Static E Millions psi	Poisson's Ratio
A	5,500	3.8	0.22
C	5,300	3.8	0.18
D	4,700	3.4	0.17
E	5,400	3.8	0.21

E, which included a 20-in. cement-treated subbase, generally had the lowest deflections and strains. Sections A and B had the highest corner and edge curl, and section E had the lowest edge curl under temperature conditions existing at the time of test.

Temperature variations are, of course, always present, and have a significant effect on the experimental data. Therefore, any quantitative comparisons between sections tested at different times must be considered in the light of such variations.

It should not be inferred that deflections such as measured during the early morning hours in sections A and B are detrimental unless it is shown that the pavement cannot sustain these deflections over a long period of time. Neither can it be inferred that section E is the best by virtue of lower deflections and strains because other factors in the future may contribute to performance.

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