Three-Year Evaluation of Shell Avenue Test Road

SHELL AVENUE TEST ROAD COMMITTEE,[1] W. A. Garrison, Chairman

The purpose of this report is to present an evaluation of performance over a 3-yr period of an experimental asphalt-concrete overlay pavement constructed on Shell Avenue in Contra Costa County, California. The overlay pavement was constructed on an existing pavement exhibiting fairly large deflections under a 15,000-lb axle load, and subjected to a large proportion of truck traffic in terms of the average daily traffic applied to the highway. Because of the existing conditions, it was planned that the test pavement should provide information on the resistance to deformation (stability) and fatigue resistance of heavy-duty mixes using conventional asphalt concrete and asphalt concrete with asbestos as a special mineral filler.

The test pavement is approximately 3,200 ft long and is divided into 4 sections, 2 with mixtures containing asbestos and 2 control sections without asbestos.

Instrumentation was installed in the pavement at the time of construction to measure dynamic deflections, bending strain, and temperature.

The report is concerned with an evaluation of periodic measurements of deflection, strain, and temperature; laboratory evaluation of cores, including density, stability as measured by the Hveem stabilometer, and viscosity at different levels in the overlay as measured by the sliding plate microviscometer; skid resistance and road roughness measurements.

From an evaluation of the field and laboratory tests, together with visual inspection of the performance of the road, conclusions are presented with regard to the ability of the various test pavements to perform under the traffic imposed and within the particular environment.

THE PURPOSE of this report is to present an evaluation of performance over a 3-yr period of an experimental asphalt concrete overlay pavement.

A previous report (1) discussed the background of circumstances which led to the decision to undertake this full-scale field investigation of ways and means of producing heavy-duty, high-quality surfacings and to explain the various levels of performance by means of physical measurements. Following an extensive period of study and planning, it was decided to limit the investigation to an evaluation of the potential benefits of using asbestos as a filler in asphalt concrete. The field test site selected for this investigation, as well as procedures used in construction, are described in some detail in that report.

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[1] Committee membership and functions are presented in Appendix A; this committee was organized to administer the Shell Avenue Test Road and is not a part of the Highway Research Board's committee structure.

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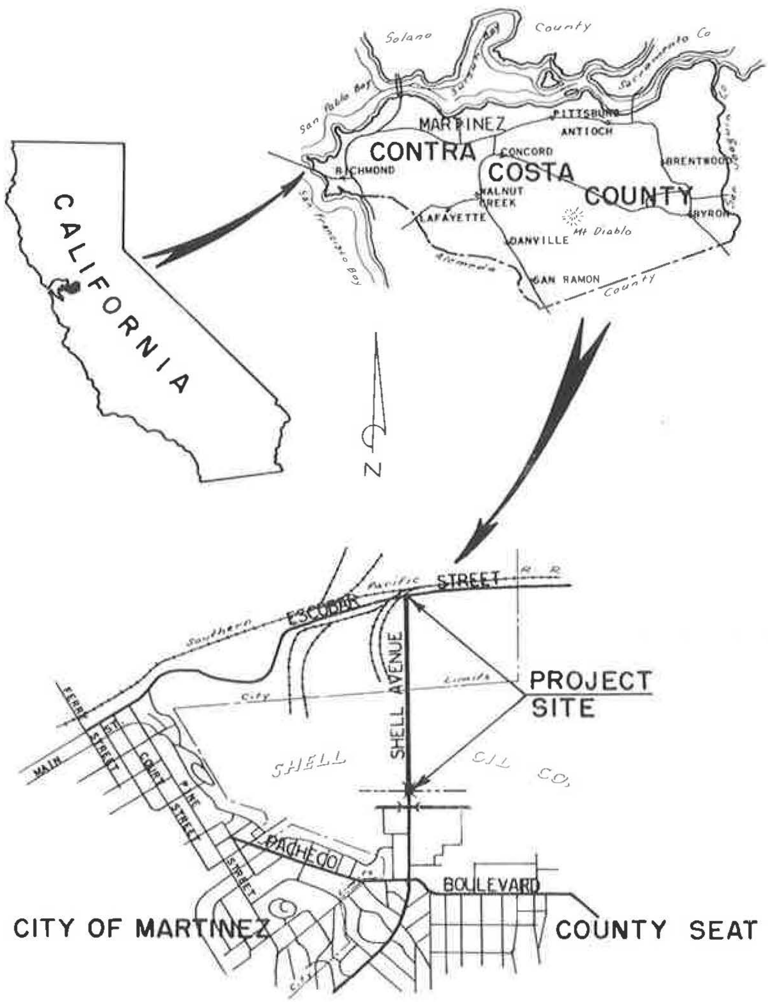

Figure 1. Site location—Shell Avenue Test Road.

Figure 2. General view of test road, before resurfacing.

Figure 3. General view of test road, after resurfacing.

filler. The experiment was designed to evaluate the relative performance of asbestos filler with 2 grades of asphalt cement when compared with comparable materials without filler.

After a year of study, during which preliminary laboratory tests and field evaluations were conducted, Shell Avenue in Martinez, California, was selected as the test road site. Specifically, an existing pavement section (Fig. 1) was selected as the test site; construction involved the placement of a nominal 3-in. resurfacing course of asphalt concrete. Shell Avenue is an industrial road carrying large numbers of heavy tank trucks and other commercial traffic. The right-of-way bisects the Shell Oil Company industrial properties. The length of the project is approximately 3, 200 ft, right-of-way is 40 ft wide, and the paved roadway section is 24 ft wide. Figures 2 and 3 are overall views of the road before and after resurfacing, respectively.

SECTION LAYOUT

Figure 4 is a schematic layout of the test project showing the location of the various test sections and describing the overlay composition. Section 4-W is composed of 40-50 penetration asphalt with asbestos filler. This section is 1, 280 ft long as compared to 640 ft for the other sections. This additional length was needed in order to include an area of high deflections found in the northern limits of the project as represented by the northernmost 640-ft section. For purposes of this report, the 4-W section was divided into two subsections of approximately equal length and designated 4-W-1 (high deflections) and 4-W-2 (normal deflections).

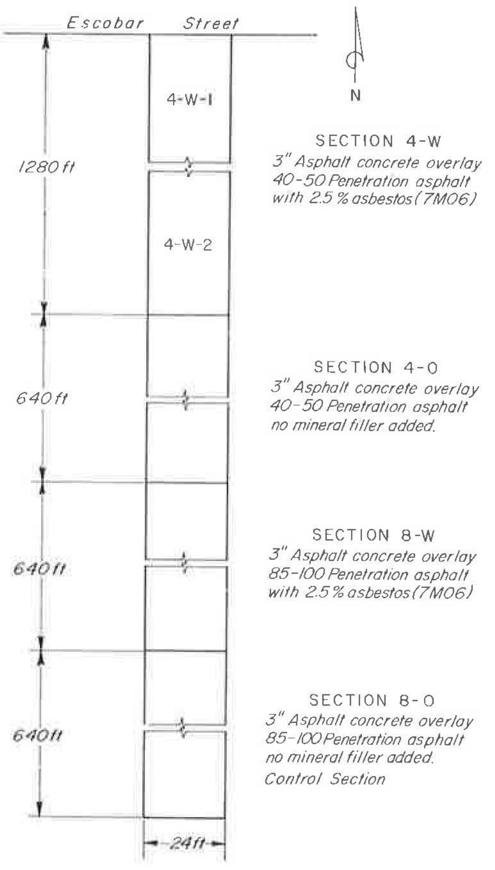

Figure 4. Schematic layout of test sections.

One of the features of this investigation which helps simplify the analysis is the manner in which traffic must operate within the limits of the project. There are no side entrances; hence, traffic must proceed through the entire length of the project, providing a continuous traffic condition for each lane. Southbound trucks were predominantly loaded, whereas northbound trucks were unloaded, making it possible to evaluate performance by lanes at 2 traffic levels. To the extent that other factors, i.e., deflection and preconstruction conditions, are similar, the difference in performance between lanes can reasonably be associated with the difference in traffic.

The 3, 200-ft test pavement was divided into 4 sections in which the nominal 3-in. asphalt-concrete overlay had the following mix proportions:

1. Dense-graded aggregate, 40-50 penetration asphalt cement, 2.5 percent asbestos fiber, asphalt content 6.7 percent. Designated Section 4-W.
2. Dense-graded aggregate, 40-50 penetration asphalt cement, asphalt content 5.8 percent. Designated Section 4-0.
3. Dense-graded aggregate, 85-100 penetration asphalt cement, 2.5 percent asbestos fiber, asphalt content 6.4 percent. Designated Section 8-W.

4. Dense-graded aggregate, 85-100 penetration asphalt cement, asphalt content 5.5 percent by dry weight of aggregate. Designated Section 8-0.

PRELIMINARY INVESTIGATIONS

Before construction of the test section, certain preliminary investigations were required, including overlay thickness design, design of the asphalt-concrete mixtures, and a survey of the condition of the existing pavement.

Thickness Design

In order to determine the desirable thickness of asphalt-concrete overlay, the California Division of Highways' method of design was utilized. Soil borings were made along the test route for evaluation of the existing subsurface materials. Hveem R-value tests were performed on samples of the underlying soil and base aggregates; a cohesiometer value was determined for the surface courses of asphalt concrete; and a future traffic index was estimated from available traffic counts. These data indicated that a 3-in. overlay would provide adequate cover in the better areas, but could be expected to be inadequate in the poorer areas. Since a certain amount of early distress in the overlaid

TABLE 1

CHARACTERISTICS OF AGGREGATE

Characteristic	Value
Specific gravity	
Coarse aggregate (3/4-in. × No. 8)	
ASTM apparent	2.89
ASTM bulk	2.84
Fine aggregate	
ASTM apparent	2.82
ASTM bulk	2.64
LA abrasion, 500 rev.	18
Sand equivalent	41

TABLE 2

CHARACTERISTICS OF ASPHALT CEMENTS

Test	Original Samples		Contractor's Storage at Time of Construction		
	85-100	40-50	85-100	40-50	40-50[1]
Pen. at 77 F	82	40	93	36	43
Ductility, cm, at 77 F	150+	150+			
Soft. Pt., R & B, F	115	128			
Viscosity					
At 77 F, poises	1.1×10^6	-			
At 140 F, poises	1393	4706			
At 180 F, poises	102.5	-			
At 275 F, stokes	2.58	4.92			
Viscosity at 275 F, SSF			122	264	
Flash point, COC, F	-	550			
Xylene equivalent			30-35	26-30	
Flash point, PMCT, F			465	-	475
Penetration ratio			30	38	
Solubility in CCl_4, %			99.9	99.9	
Thin film oven test					
Loss on heating, %			0.41	0.46	
Ret. of pen., %			60	67	
Duct. of residue			111+	111+	
Specific gravity, 77/77 F	1.015	1.020			

[1]From supplier's storage at time of shipment.

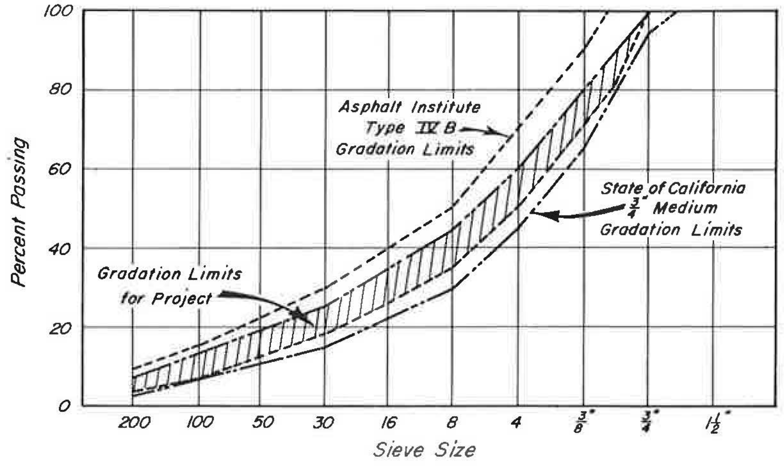

Figure 5. Aggregate gradation limits.

TABLE 3

MIX PROPERTIES AT DESIGN ASPHALT CONTENTS

Mix No.	Asphalt Content (% of Dry Wt of Agg.)	Relative Stability (S)	Cohesiometer Value (C)	Unit Weight (pcf)	% Voids Total Mix
8-0	5.5	45	340	154.5	5.2
8-W	6.4	43	400	151.8	5.4
4-0	5.8	49	600	154.1	5.2
4-W	6.7	48	520	151.8	5.2

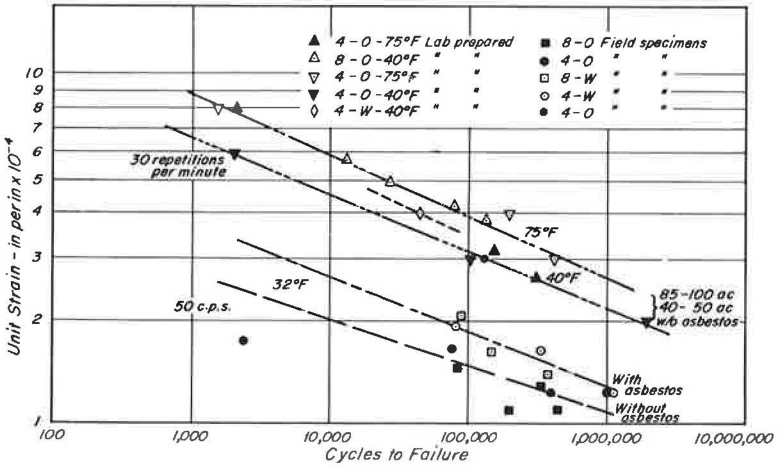

Figure 6. Fatigue test results on laboratory and field-compacted specimens.

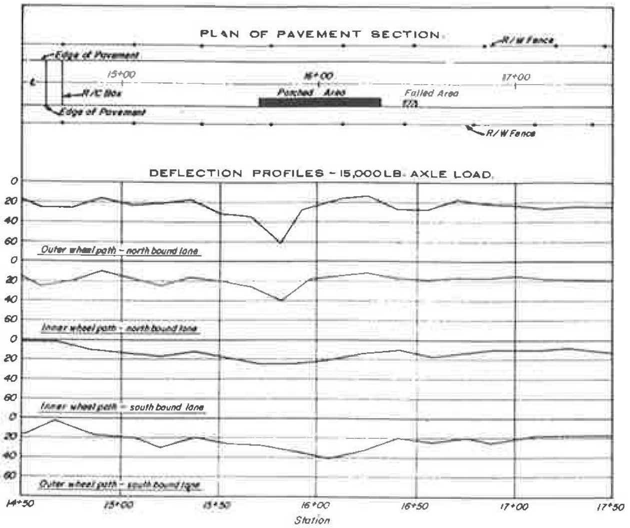

Figure 7. Typical deflection profile—low deflection area section 4-0.

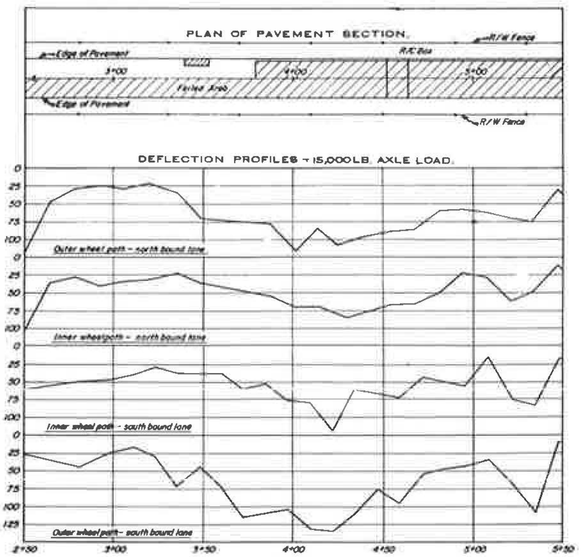

Figure 8. Typical deflection profile—high deflection area section 4-W-1.

pavement would contribute to the success of the experiment, it was decided to overlay the existing pavement with 3 in. of asphalt concrete, placed in 2 equal lifts.

Mix Design

Four different mixtures were utilized in the project. The aggregate for all of the mixtures was a crushed basaltic type material obtained from a local quarry; it has an excellent service record in pavements in the area. Typical test properties of this material are given in Table 1.

Figure 9. Typical pavement failure.

Figure 10. Typical pavement failure.

Figure 11. Typical pavement failure.

Figure 12. Typical pavement failure.

TABLE 4

SUMMARY OF ALL CRACKING IN EXISTING PAVEMENT PRIOR TO RESURFACING

Section	Area (sq ft)	Percent
4-W-1	8842	80.3
4-W-2	6398	48.2
4-0	345	4.5
8-W	533	5.0
8-0	1435	13.3

The asphalts used are from California crudes as produced and supplied by the Shell Oil Company. Results of standard tests on both the 85-100 and 40-50 materials are given in Table 2.

In order to comply with both The Asphalt Institute specifications Type IV-b and the California Division of Highways ¾-in. maximum medium gradations, the overlapping portion of these gradation bands was utilized as the limits for the asphalt concrete used for the project. The specification limits are shown in Figure 5; this gradation specification is more restrictive than either of the specifications named.

The design asphalt content for each of the mixes was selected on the basis of tests conducted by The Asphalt Institute laboratory at College Park, Md. Recommended asphalt contents, and also mix properties for each design, are given in Table 3. Values were selected on the basis of results of both the Hveem and Marshall stability tests.

While not a part of the actual mixture design, constant-strain amplitude fatigue tests were conducted on laboratory-prepared beam specimens of the mixes without asbestos at the design asphalt contents and at approximately the densities obtained during field compaction (Fig. 6). Results of tests on one series of specimens containing the 40-50 penetration asphalt and asbestos, also conducted at 40 F, are also shown in Figure 6.

These tests were performed with apparatus described elsewhere (2) at a frequency of loading of 30 applications per min and a duration of loading of 0.1 sec. For purposes of comparison, test results for a series of slabs sawed from the 4-0 section of pavement immediately after construction are also presented. Essentially the same fatigue life at 300×10^{-6} in./in. strain is obtained for both the laboratory-prepared and field-compacted specimens.

An additional series of constant-stress amplitude fatigue tests was performed on field specimens at 32 F and a frequency-of-stress application of 50 cps. These data are presented in Figure 6. Essentially the same trends were obtained in these tests on the field specimens as were obtained on the laboratory-prepared specimens.

Initial Deflection and Crack Survey

A condition survey was made of the existing pavement by measuring deflections throughout the length of the project and by conducting a crack survey. Deflections were measured in both the inner and outer wheelpaths of the northbound and southbound lanes using the traveling deflectometer developed by the California Division of Highways. Except for the first approximately 600-ft length of the project (the northern section), the initial deflections were relatively uniform. A representative deflection for a section of this latter (major) portion is shown in Figure 7. For comparison, deflections from the northern 600 ft are shown in Figure 8. Deflections as high as 0.135 in. were obtained in this area. Areas which were patched or considered to be failed as a result of the crack survey are also shown (Figs. 7 and 8). Figures 9 through 12 show typical examples of the failed areas.

The initial crack survey was conducted by outlining the cracked areas and converting them to square feet of cracking. Table 4 summarizes all cracking present in the existing pavement before resurfacing.

INSTRUMENTATION

To assist in the interpretation of the performance data obtained from the test road, instrumentation was installed in the pavement at the time of construction to measure dynamic deflections, bending strains, and pavement temperature. This instrumentation consisted of linear variable differential transformers, variable-resistance bonded wire strain gages, and thermocouples.

A typical linear variable differential transformer (LVDT) installation is shown in Figure 13. Four such installations were constructed in each test section. Although fixed in position (a possible disadvantage), these gages have the advantage, when compared to the Benkelman beam, of being able to determine the complete deflection profile for any tire configuration, and for deflections of the pavement under rapidly-moving wheel loads.

The bonded wire strain gages were installed to provide data on the bending strains induced in the resurfacing by moving wheel loads. These gages were installed on top of the existing surface prior to resurfacing near each of the LVDT installations. Two gages were placed at a specific location, one oriented parallel to and the other normal (or transverse) to the direction of traffic. After the resurfacing had been placed, two gages were also installed on the pavement surface in approximately the same locations as the gages bonded to the existing pavement. By placing both sets of gages near the LVDT installations, the radius of curvature of the deflected surface as determined from the LVDT could be related to the measured bending strains. Typical recordings of deflection and strain are shown in Figure 14 for a 15,000-lb axle load moving at creep speed at gage point 18.

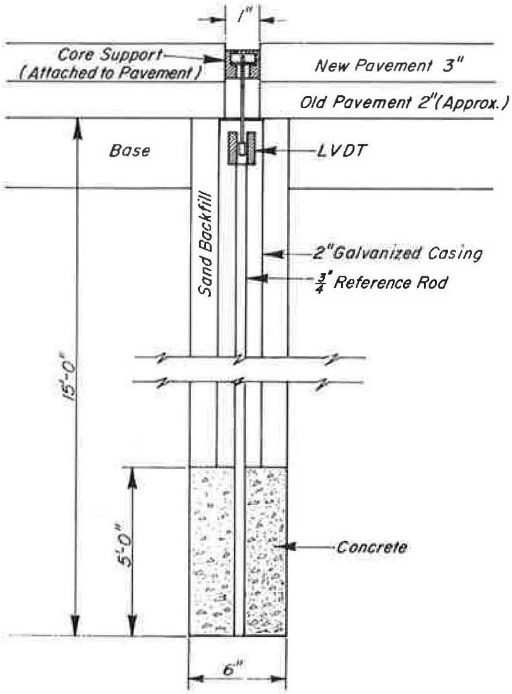

Figure 13. Typical linear variable differential transformer (LVDT) installation.

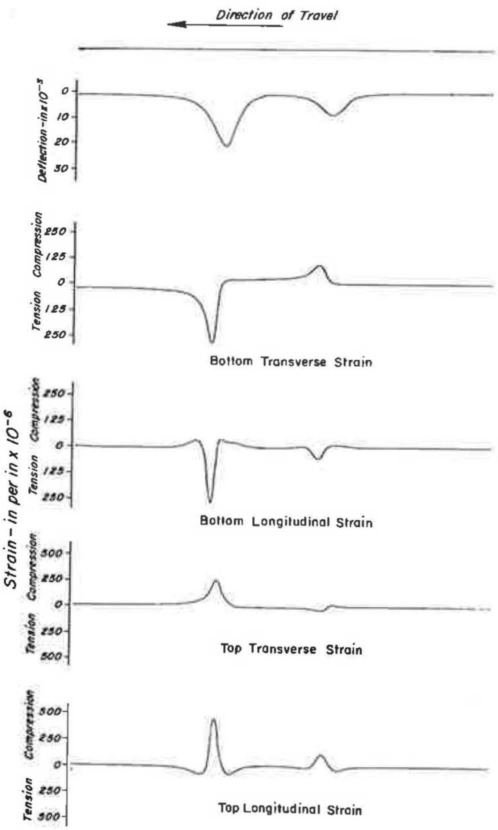

Figure 14. Oscillogram illustrating strain and deflection variation with passage of a 15,000-lb single-axle load at gage point 18.

As noted, one of the objectives of the test road project was to study the resistance of heavy-duty mixtures to fatigue cracking. Current evidence would indicate that the magnitude of the tensile strain repeatedly applied appears to be a satisfactory criterion for ascertaining the development of fatigue cracking. The strain gages thus provide a direct measure of strains occurring in the pavement. It was hoped that these measured values could be related to laboratory-determined values such as those in Figure 6.

Since temperature has an effect on the flexural stiffness of asphalt concrete, thermocouples were installed in each section to measure the temperature near the surface, at middepth, and at the bottom of the overlay. Temperature measurements were made each time the deflection and strain measurements were obtained.

CONSTRUCTION MEASUREMENTS

In order to have a more complete record of initial properties of the asphalt concrete as placed, and of construction procedures and conditions, a number of special tests were made.

Table 5 gives the results of water permeability tests made on the completed resurfacing approximately 20 hr after construction. Equipment and techniques used are as developed by the California Division of Highways and described in detail in Test Method No. Calif. 341-A. A tentative limit of 150 ml/min has been suggested (3) as a maximum acceptable permeability, with consideration being given to use of a seal coat on pavements with measured permeability greater than this limit.

Results of air permeability tests are given in Table 6. Equipment and techniques used for conducting this test are those developed by the California Research Corporation (4). No limiting values of air permeability have been suggested for general use.

TABLE 5

WATER PERMEABILITY DATA 20 HOURS AFTER CONSTRUCTION

Test Section	Water Permeability (ml per min)							
	Leveling Course				Surface Course			
	OWP[1]	BWP[1]	IWP[1]	Avg	OWP	BWP	IWP	Avg
4-W	20	25	105	64	15	14	25	16
4-0	160	180	255	206	45	25	25	33
8-W	38	33	42	38	34	28	25	29
8-0	38	35	55	40	40	28	48	38

[1] OWP—Outer Wheelpath; BWP—Between Wheelpaths; IWP—Inner Wheelpath.

TABLE 6

AIR PERMEABILITY DATA IMMEDIATELY AFTER CONSTRUCTION

Test Section	Air Permeability (ml per min at 0.25-in. pressure)			
	Northbound		Southbound	
	Average	Range	Average	Range
4-W	21	1-118	104	55-150
4-0	162	13-428	499	188-811
8-W	19	3-79	187	112-289
8-0	101	40-189	137	75-191

TABLE 7

REPRESENTATIVE MIX TEMPERATURES DURING CONSTRUCTION

Test Section	Temperature (F)		
	Placing	Initial Rolling	Pneumatic Rolling
4-W	231	206	162
4-0	266	196	180
8-W	221	213	189
8-0	233	221	179

TABLE 8

SUMMARY OF TEST RESULTS ON CORES TAKEN IMMEDIATELY AFTER CONSTRUCTION

Test Section		Unit Weight (pcf)		% Air Voids (avg)	Individual Cores					
					Hveem Tests			Marshall Tests		
		Avg	Range		S	C	Unit Wt (pcf)	Stab. (lb)	Flow (0.01 in.)	Unit Wt (pcf)
4-W	N	152.7	152.5-153.3	1.6	23	405	152.5	2622	43	152.5
	S	148.6	147.8-149.3	4.2	18	282	149.3	1922	54	148.1
4-0	N	152.4	150.8-153.4	3.0	26	247	153.4	1584	30	153.1
	S	149.9	149.2-151.1	6.0	18	137	149.1	1176	26	150.1
8-W	N	152.3	151.8-153.0	2.2	17	409	153.0	1488	28	151.9
	S	148.6	146.0-149.4	4.9	15	246	147.9	895	28	149.4
8-0	N	155.5	154.4-156.4	1.4	20	214	155.8	1950	29	156.4
	S	153.7	153.2-154.4	2.6	23	212	153.6	1406	22	154.4

Thermocouples were installed in the mix at the time of construction to develop information on mix temperatures during placing and rolling. Table 7 gives representative temperatures at approximately ¾ in. below the surface as measured by these thermocouples during construction operations.

Immediately following completion of the resurfacing, cores were cut at selected locations in each test section. Test results are summarized in Table 8. Extraction tests were conducted on samples of the asphalt concrete obtained at the hot-mix plant during production. Aggregate gradation and asphalt content as determined from these samples are given in Table 9 and compared with specified gradation limits and design asphalt content.

TABLE 9

RESULTS OF EXTRACTION TESTS ON PLANT SAMPLES

Sieve Sizes	Percent Passing				
	Specification Limits	Section 4-W	Section 4-0	Section 8-W	Section 8-0
¾ In.	100	100	100	100	100
½ In.	-	90	85	88	88
⅜ In.	70-80	76	64	68	66
No. 4	50-60	64	54	56	54
No. 8	35-45	42	35	41	35
No. 16	-	30	26	32	25
No. 30	18-25	23	22	27	20
No. 50	-	16	18	20	16
No. 100	-	10	10	11	10
No. 200	4-7	5.8	6.8	6.3	6.8
Asphalt Content[1]		7.0	5.3	6.0	4.9
Design Asphalt Content[1]		6.7	5.8	6.4	5.5

[1] Percent of dry weight of aggregate.

The average measured thickness of asphalt-concrete resurfacing, calculated from measurements made on 111 cores taken during the entire period of study, was 3.04 in.

TEST PROGRAM

To measure properly changes occurring in the various test sections and to attempt to relate these changes to traffic and environment during the test period, a comprehensive series of field and laboratory measurements was planned. The test program included, where appropriate, the following measurements before and after pavement construction:

1. Traffic and load surveys;
2. Deflection measurements with the traveling deflectometer of the California Division of Highways;
3. LVDT deflection and strain measurements at permanent gauge installations;
4. Tests on cores for determination of changes in mix and asphalt properties with time;
5. Precise levels;
6. Road surface measurements including skid resistance and road roughness;
7. Condition (cracking) surveys by visual observation.

Traffic Counts and Index

The 1964 California Division of Highways procedures were followed in evaluating traffic characteristics on the project during the initial 3-yr period following construction. To convert traffic into a traffic index, it was necessary to have some knowledge of truck traffic, axle configuration, and load.

Visual truck traffic counts, including axle configuration, were made at 4 different times through July 1964. These counts were made during the weeks of Aug. 25-Sept. 1, 1961; Aug. 27-31, 1962; Sept. 24-28, 1962; and March 4-7, 1963. The computations showed that traffic was essentially constant for the first 3 traffic counts. The final count was made after the opening of the Benicia-Martinez Bridge which permanently rerouted certain truck traffic from the test road to a new highway. This last count was substantially lower than on previous dates; however, it is believed to be representative of present and future traffic.

Typical gross load information (loaded and unloaded) was recorded as part of the traffic count, and was used to determine an equivalent wheel load factor (loaded and unloaded) for 2-, 3-, 4-, and 5-axle trucks.

To determine equivalent wheel load from gross load data, it was necessary to distribute the load to the various axles. A typical axle configuration for each type of truck was assumed, based on what is believed to be the most common truck design operating in the area. The assumed load distribution between axles is based, to a large extent, on information on truck weights from the AASHO Road Test. Both of these assumptions could be subject to some variation. It is believed, however, that the assumptions made will result in a reasonable calculation of average equivalent wheel load factors.

With the loads for each wheel determined, it was possible to convert to an EWL factor using the 1963 equations of the California Division of Highways. On the basis of these load factors, the traffic was converted to equivalent 5000-lb wheel loads for northbound and for southbound traffic. The traffic index was then computed from the following formula:

$$TI = 6.7\left(\frac{EWL}{10^6}\right)^{0.119}$$

The estimated southbound traffic index is 6.9 and the northbound traffic index 6.3 for the 3-yr period.

Cracking Surveys

Pavement cracking surveys were made at 4 intervals from the inception of the project to the end of the study period. The initial survey made in March 1961 (before the overlay) represents the preconstruction condition and was discussed above. Subsequent crack surveys were made in March 1962, August 1962, March 1963, and June 1964.

To show the progressive manner in which cracking developed, cumulative percent of pavement cracked, by lane and wheelpath, is given in Table 10. This tabulation is simply the length in which cracking was present, expressed as percent of total length.

For purposes of analysis, the results of the preconstruction and June 1964 cracking surveys only were used, and a refinement in the measurement of cracking was made. Table 11 gives these results. Since cracking is expressed here as percent of total area, rather than length, values shown here cannot be compared with those in Table 10.

It is important to point out that certain limitations in construction did not allow precise control of placing of the various types of materials within established boundaries; therefore, 100-ft transitions between sections were eliminated from the analysis.

TABLE 10

PROGRESSION OF CRACKING AS SHOWN BY RESULTS OF THREE POST-CONSTRUCTION SURVEYS

Section	Date	Cumulative Percent of Pavement Length Cracked				
		Southbound Lane		Northbound Lane		Total for Section
		IWP	OWP	IWP	OWP	
4-W-1	Mar 1962	4	0	0	7	3
	Oct 1962	18	45	27	42	33
	Mar 1963	56	45	31	45	44
4-W-2	Mar 1962	47	0	0	0	12
	Oct 1962	47	2	0	0	12
	Mar 1963	49	3	26	3	20
4-0						(No cracking)
8-W	Mar 1962	11	0	0	0	3
	Oct 1962	13	0	1	0	3
	Mar 1963	19	1	2	7	7
8-0						(No cracking)

TABLE 11

COMPARISON OF CRACKING MEASURED IN JUNE 1964 WITH PRECONSTRUCTION CRACKING

Section	Date	Percent of Pavement Area Cracked				
		Southbound Lane		Northbound Lane		Total for Section
		IWP	OWP	IWP	OWP	
4-W-1	Preconstr.	98	98	68	57	80
	June 1964	20	16	15	18	17
4-W-2	Preconstr.	50	55	37	52	48
	June 1964	31	11	9	10	15
4-0	Preconstr.	0	18	0	0	5
	June 1964	0	0	0	0	0
8-W	Preconstr.	7	13	0	0	5
	June 1964	12	9	2	2	7
8-0	Preconstr.	30	18	6	0	13
	June 1964	0	0	0	0	0

The cracking survey made in June 1964 resulted in the identification of two important crack patterns. The predominant type of crack is longitudinal, occurring generally within the limits of the wheelpath. Some other cracking, often referred to as "chicken-wire" or "pattern" cracking, was also observed. Both longitudinal and chicken-wire cracking were plotted regardless of the degree or amount of progression (i.e., "hair-line" cracking was included), and the area reported was based on the presence or absence of cracking, without regard to degree or severity. In some instances, where cracking is just barely discernible, this criterion of performance could be considered as a very severe judgment of distress. However, it is likely that these hairline cracks are signs of impending distress, given enough time and traffic.

Deflection Surveys with Traveling Deflectometer

Pavement deflection surveys were made with the California Division of Highways traveling deflectometer (5). This equipment measures surface deflections under a 15,000-lb single-axle load and provides an analog trace of the deflected basin along the longitudinal axis.

Deflection measurements were made with this equipment 5 times up to June 1964. These deflection runs were made on: March 27, 1961 (preconstruction); Sept. 20, 1961; March 8, 1962; Aug. 31, 1962; and March 18, 1963. Since the parameters of the deflection test are considered of value primarily as predictors of future performance, it was desirable to select data from earlier tests (i.e., either the Sept. 1961 or March 1962 series of measurements) as a basis for the analysis. The March 1962 run was selected for 2 reasons:

1. The Sept. 1961 run was taken too soon (approximately one week) after construction to allow the mix to assume a condition representative of its long-term characteristics.
2. There was less scatter in the data of March 1962, indicating somewhat more reliable information.

Average deflections for Sept. 1961 and March 1962 for the southbound lane are shown separately by section and wheelpath in Figure 15. Similar data for the northbound lane are shown in Figure 16. Although the Sept. 1961 deflection measurements are not used in the analysis, they are included in these figures to illustrate the considerable reduction in deflections from immediately following construction to only a few months later.

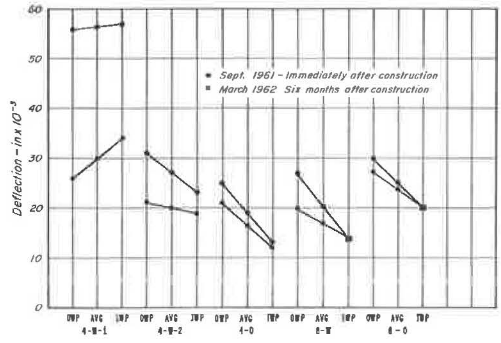

Figure 15. Average deflections by section and wheelpath for the southbound lane.

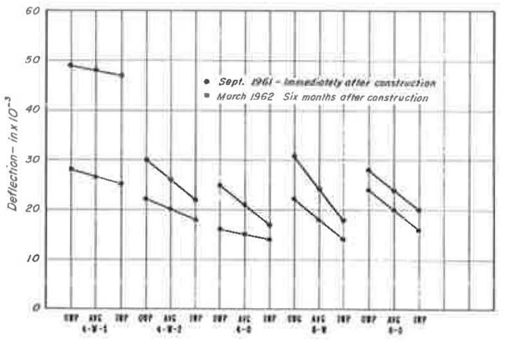

Figure 16. Average deflections by section and wheelpath for the northbound lane.

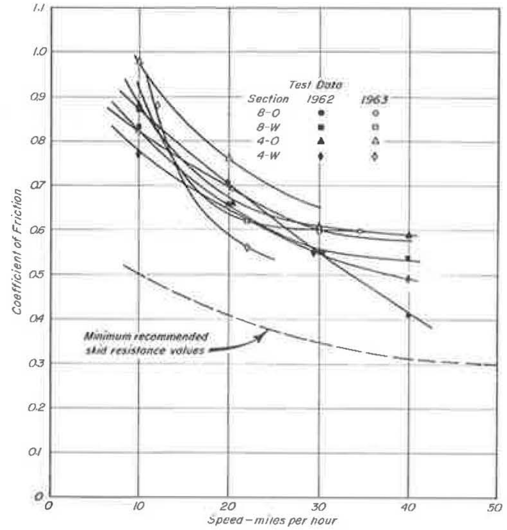

Figure 17. Summary of 1962 and 1963 skid resistance measurements—wet pavement tests.

Surface Measurements

Skid resistance tests were performed on each of the test sections in March 1962 and Dec. 1963. Tests were conducted with the University of California skid resistance equipment using a 1958 Standard Test Tire (6). Values obtained for coefficient of friction for all sections (Fig. 17) are in the range considered to give skid resistance comparable to well-constructed State Highways in California.

Road roughness tests were performed in Nov. 1961, using University of California equipment (7). The roughness index at 20 mph was 106 in./mi for the southbound lane and 102 in./mi for the northbound lane. Good riding quality is indicated since values less than 125 in./mi are considered satisfactory for this type of facility.

Precise level surveys were made on the surface immediately following placing of the resurfacing and periodically thereafter. Comparison of the results of these surveys indicates no measurable rutting or distortion. Comparison of density measurements made on cores taken immediately after construction with those taken in 1962 and 1963 indicates no significant trend toward densification under traffic. This tends to verify conclusions made from the precise level surveys.

Deflection (LVDT) and Strain Data

Table 12 summarizes deflection, strain and temperature data obtained from the deflection and strain gage installations in the various sections during the period Oct. 1961 to April 1963. Since the measurements covered periods of time of as much as an hour at a particular gage point, the temperature range during this interval is given in many instances.

At a particular point, measurements were taken both of strain and deflection for various positions of the wheel load with respect to the gage installation and with the truck (single rear-axle, dual tires, 15,000-lb axle load) traveling at creep velocity (1 to 3 mph). These measurements at the various positions permit development of

TABLE 12

SUMMARY OF DEFLECTION AND STRAIN MEASUREMENTS AT FIXED GAGE INSTALLATIONS RESULTING FROM 15,000 LB AXLE LOAD ON DUAL TIRES TRAVELING AT CREEP SPEED (1—3 MPH)

Section	Gage Point	Date	Air Temp. (F)	Pavement Temperature (F)			Maximum Deflection (in.)	Maximum Observed Tensile Strain (in. per in. × 10^{-6})			
								Bottom		Top	
				Bottom	Middle	Top		Transverse	Longitudinal	Transverse	Longitudinal
4-W-1	1	17 Apr 62	-	95-102	106-114	116-121	0.040	35	*	150	100
	4	17 Apr 62	-	102-104	112	120-116	0.056	60	*	120	75
	4	28 Aug 62	-	101-111	124-140	127-134	0.056	40	*	230	100
4-W-2	5	26 Oct 61	-	74-76	76-78	78-84	0.025	25	15	25	25
	5	27 Dec 61	44	47-50	48-50	49-51	0.0185	40	30	50	20
	5	27 Dec 61	44	48	49	50	0.018	65	60	15	5
	5	17 Apr 62	73-81	83-93	90-103	102-119	*	40	10	90	70
	5	28 Aug 62	-	105-107	115-116	129-124	0.036	100	95	145	235
	6	1 Nov 61	-	63	62	62	0.018	30	80	75	65
	6	27 Dec 61	45	48	48	50	0.018	90	75	10	20
	6	18 Apr 62	-	74-78	76-82	78-86	0.014	400	*	*	65
	6	29 Aug 62	-	76-100	76-108	78-119	0.0205-0.023	475	*	130	250
	7	17 Apr 62	-	99	109-102	118-107	0.060	80	110	125	70
	7	12 Apr 63	-	78	84	90-92	0.0275	70	105	40	40
4-0	9	26 Oct 61	-	76	78	78-77	0.028	25	75	75	50
	9	22 Nov 61	-	56	55	55	0.0165	20	25	25	15
	9	26 Dec 61	56	60	64	66	0.022	20	20	15	25
	9	27 Dec 61	43-40	50	48	46	0.016	25	25	25	35
	9	18 Apr 62	-	80	83	86	0.033	15	80	85	45
	9	30 Aug 62	-	77-79	76-80	77-84	0.025	70	75	70	50
	9	12 Apr 63	-	85-80	90-89	90-88	0.021	35	115	40	60
	10	18 Apr 62	-	85-91	90-98	97-105	0.060	35	400	95	90
	10	12 Apr 63	72-70	-	82-88	92-86	0.032	225	450	35	50
8-W	14	25 Oct 61	-	67-78	70-84	75-86	0.041	20	15	175	45
	14	22 Nov 61	-	56	55	55	0.0235	5	30	40	15
	14	27 Dec 61	46	54	55	55	0.025	10	20	50	40
	14	18 Apr 62	-	96	103	103	0.041	90	145	70	150
	14	31 Aug 62	-	80-84	82-100	88-104	0.027	25	50	140	150
	14	13 Apr 63	70-73	70	80-84	84-85	0.019	*	45	65	65
	15	26 Oct 61	-	68-72	74-80	78-82	0.0435	0	*	90	40
	15	26 Dec 61	-	60	64	67	0.0395	15	45	50	50
	15	27 Dec 61	40	48	46	44	0.0335	20	25	40	40
	15	19 Apr 62	-	94-91	99-89	100-84	0.054	0	60	215	65
	15	30 Aug 62	-	98-100	111	113-117	0.0425	70	30	415	175
	15	13 Apr 63	74	71-78	80-84	85-88	0.029	40	*	90	75
	16	26 Oct 61	-	74	78	81	0.033	*	*	100	150
8-0	18	25 Oct 61	-	79	-	80	*	170	280	115	100
	18	26 Dec 61	52	62	-	-	*	60	125	525	150
	18	27 Dec 61	-	45	-	40	*	80	230	25	15
	18	19 Apr 62	-	94	-	-	0.043	575	500	150	240
	18	30 Aug 62	-	114	-	120	0.037	365	420	450	150
	18	31 Aug 62	-	74-82	-	78-84	0.0255	220	285	100	75
	18	13 Apr 63	70-73	70	80-84	84-85	0.023	350	345	115	175
	19	24 Oct 61	-	86-88	-	-	0.036	*	*	150	140
	19	26 Dec 61	46	50-53	-	-	0.0265	*	*	40	200
	19	27 Dec 61	48-46	54-56	-	-	0.025	*	*	140	125
	19	27 Dec 61	39	45	-	-	0.0205	*	*	125	40

*Gage inoperative.

complete patterns of deflection and strain at the gage point. In addition to the series of measurements at creep speed, additional measurements of deflection and strain were obtained for velocities of up to 40 mph for passage of the center of the duals of the rear axle over the gage point.

The data in Table 12 were obtained from the creep speed measurements. Maximum measured values for deflection and tensile strain are listed to permit a comparison between sections. In general, the highest values of strain were recorded in Section 8-0. Comparing gage points 18 (Section 8-0) and 15 (Section 8-W) on April 13, 1963, for example, with about the same pavement temperatures in both instances, the observed tensile strains at gage point 18 are considerably larger than those at point 15.

Many different analyses can be made for the strain and deflection data. Some of the possibilities are included to give an idea of what can be done rather than to establish definite criteria.

Figure 18 shows the complete deflection pattern of the pavement at gage point 18 on April 19, 1962. The maximum deflection occurs under one of the tires in this instance. These data were developed from the recordings of deflections obtained by moving the loaded wheel with respect to the gage installation. Figures 19 and 20 illustrate the variation of longitudinal strain both at the top and underside of the pavement for the same conditions.

At the time these measurements were being obtained, a 5-axle truck with a gross load of approximately 75,000 lb also passed over the gage installation. The recording (Fig. 21) was made with the centerline of the dual tires passing over the point; therefore

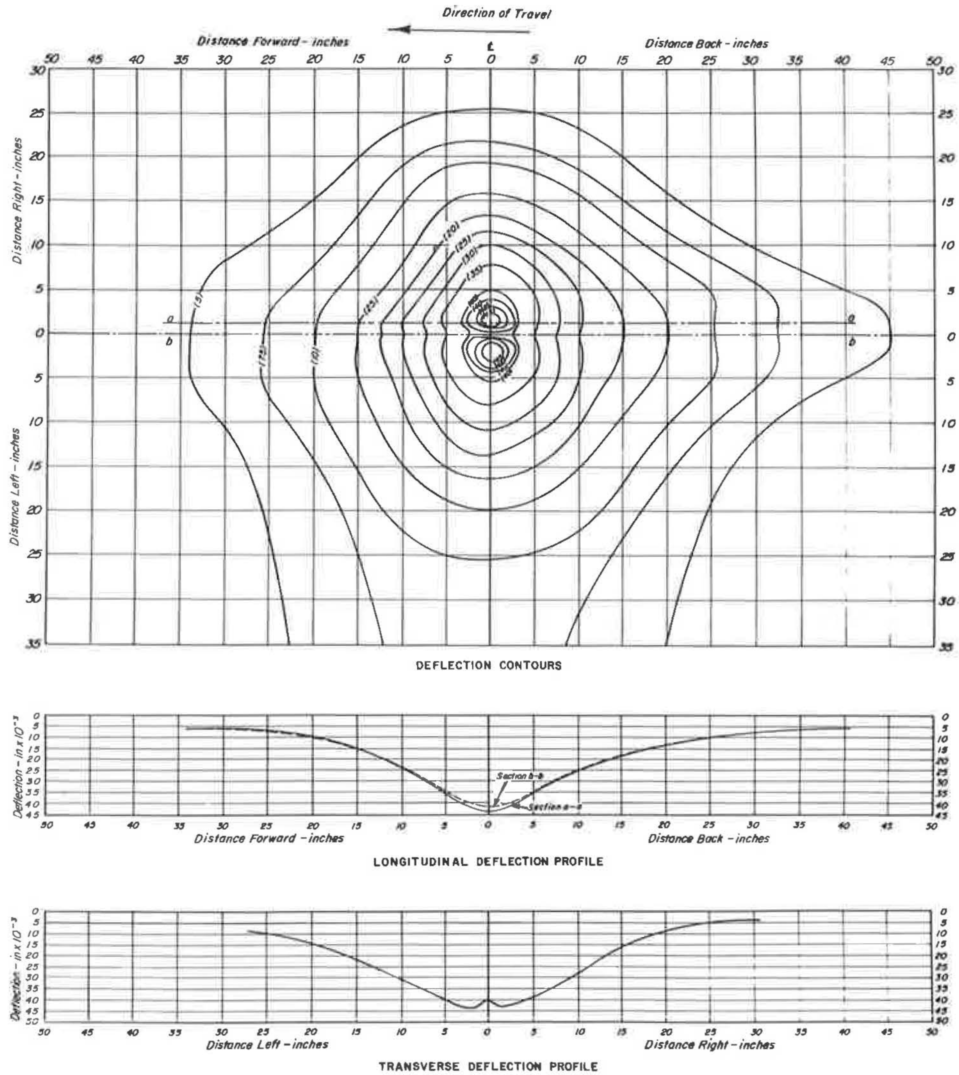

Figure 18. Deflection contours and profiles at gage point 18, April 1962—15,000-lb single-axle load on dual tires, creep speed.

a comparison between the deflection profile for this truck and the 15,000-lb axle load normally used could be obtained (Fig. 22). Although the deflection under the rear axle of the truck is actually larger, the shape of the deflection curve would indicate no more severe strains than those developed for the test vehicle. Of interest in Figure 21 is the high tensile strain developed by the front axle in this instance. At times the front axle is neglected in pavement design evaluation. This measurement, along with analysis of many of the recordings from this project, indicate that often the front axle is at least as severe as the rear axle in terms of inducement of strain.

As noted earlier, the effect of vehicle speed on deflection and strain was obtained by passing the center of the rear duals of the vehicle over the gage point at speeds up to 40 mph. Figure 23 shows the reduction in deflection with increase in speed at point 18 on April 19, 1962. At this time, the temperature at the bottom of the overlay pavement was of the order of 97 F. A reduction of almost 0.008 in. was obtained over the

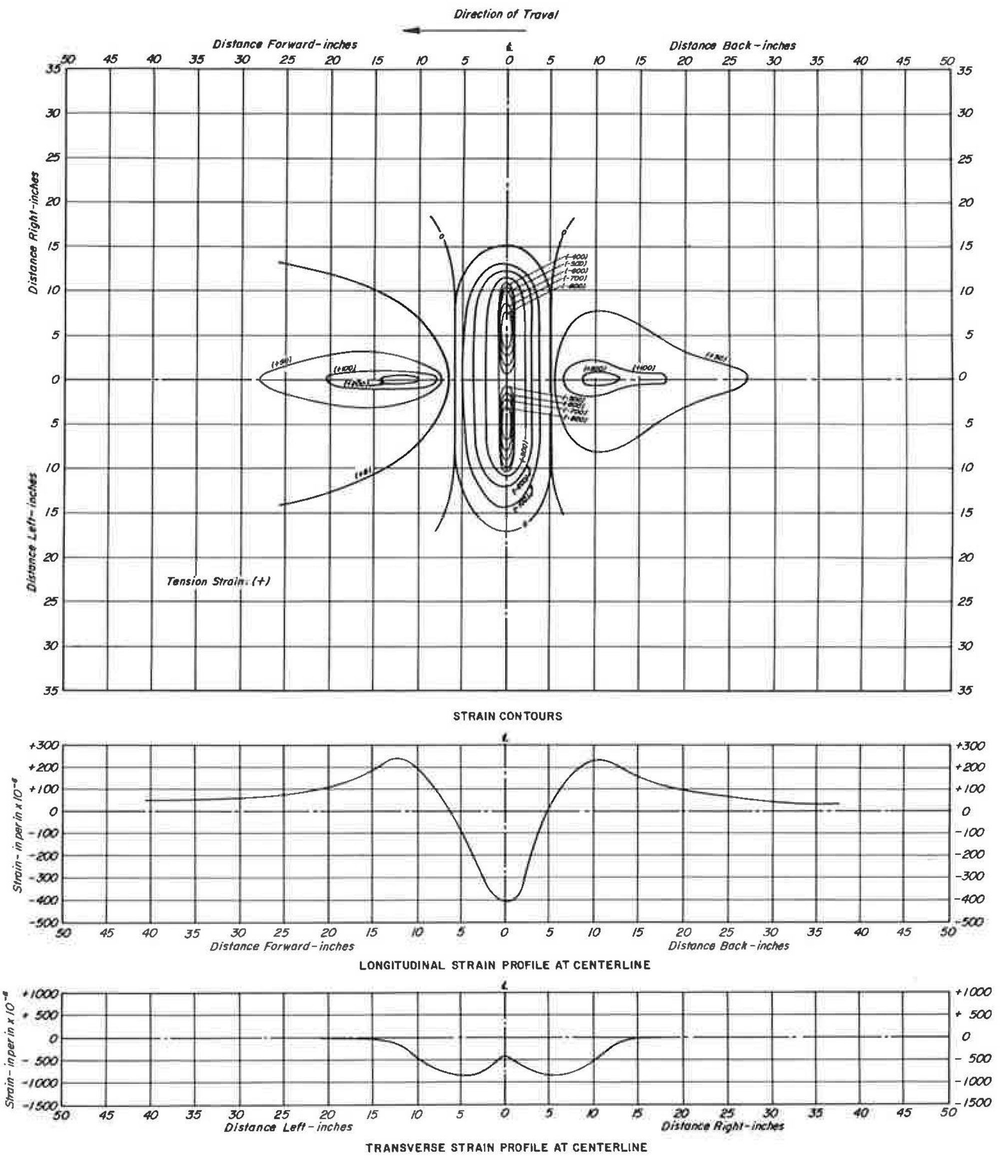

Figure 19. Contours and profiles of top longitudinal strain at gage point 18, April 1962—15,000-lb single-axle load on dual tires, creep speed.

range of vehicle speeds investigated. One cannot, of course, always guarantee this much change. Figure 24 shows, for the 4-0 section, the effects of both speed and temperature on deflection. At the lesser temperatures the reduction in deflection with increased speed is comparatively small.

The shape of the deflected surface is considerably affected by temperature. Figure 25 shows deflection profiles normal to the direction of travel at gage point 18 obtained in August 1962. The effect of mixture stiffness is quite apparent.

One of the interesting, and perhaps significant, measurements in the field investigation is the transverse strain. Figures 25 and 26 show the transverse deflection profile and variation of transverse strain both in the top and the underside of the overlay.

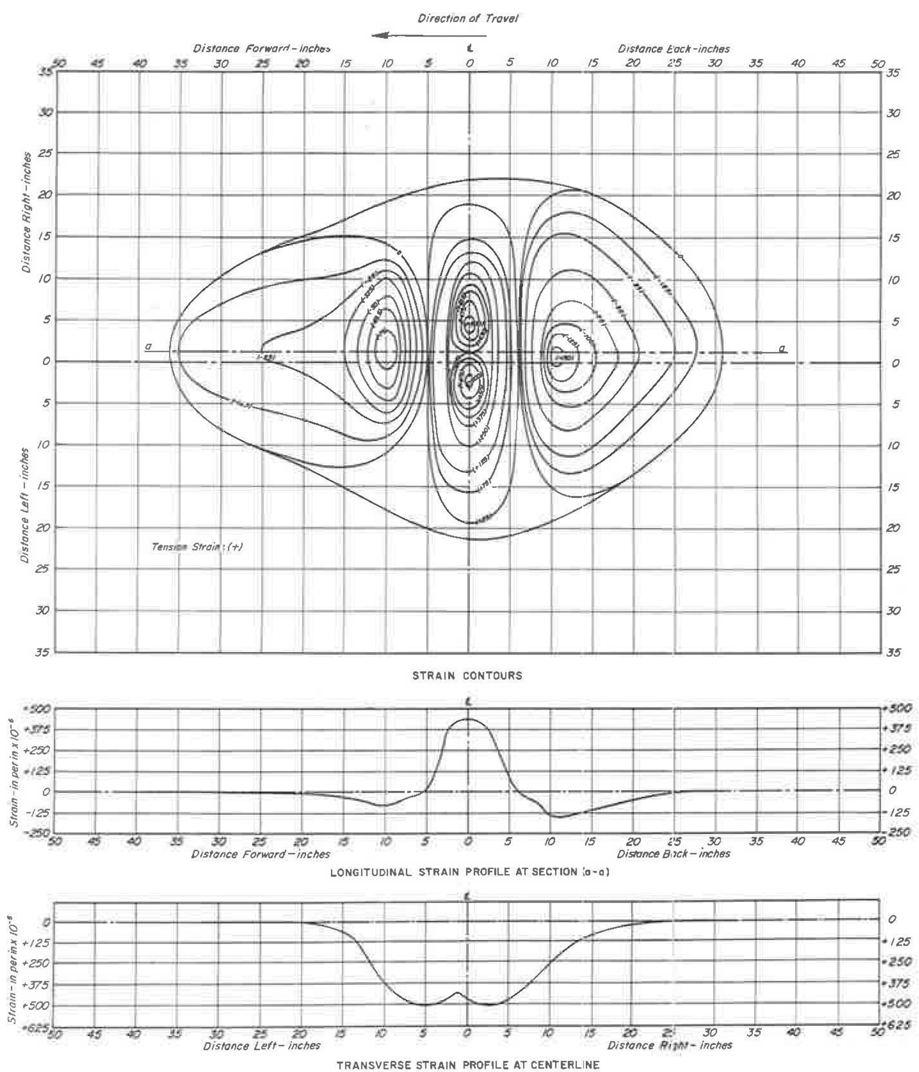

Figure 20. Contours and profiles of bottom longitudinal strain at gage point 18, April 1962-15,000-lb single-axle load on dual tires, creep speed.

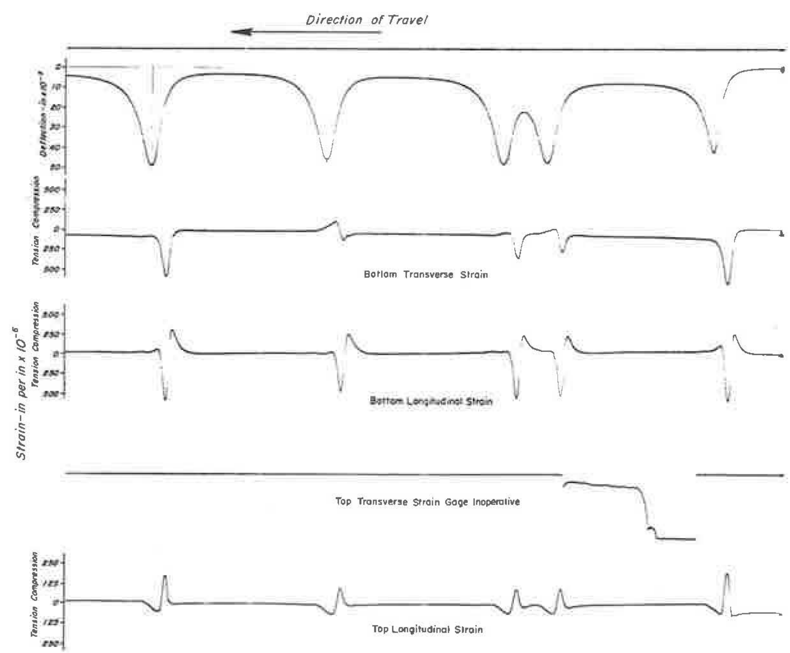

Figure 21. Oscillogram illustrating strain and deflection variation with passage of 5-axle truck with a gross load of 75,000 lb over gage point 18.

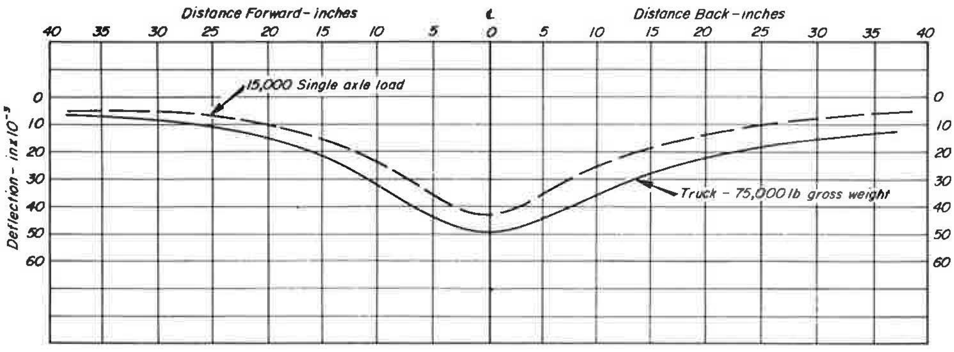

Figure 22. Comparison of centerline longitudinal deflection profiles for 15,000-lb single-axle load and rear axle of 5-axle 75,000-lb gross load truck at gage point 18.

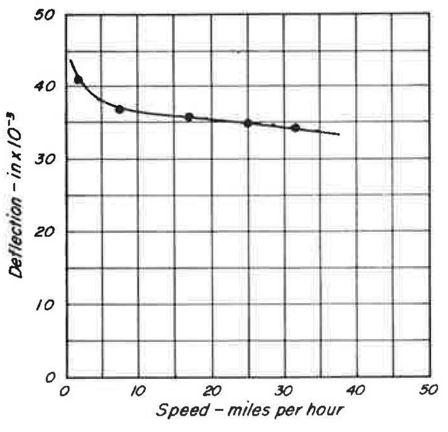

Figure 23. Effect of speed on centerline deflection for 15,000-lb single-axle load at gage point 18, April 1962.

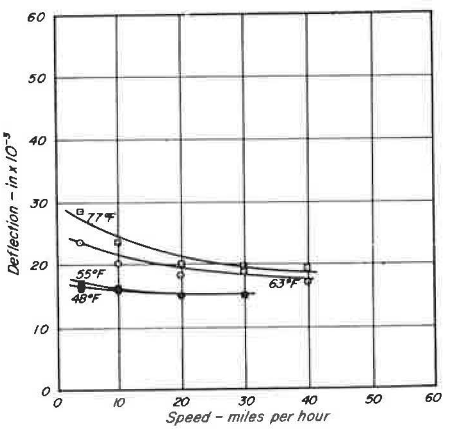

Figure 24. Effect of speed and temperature on centerline deflection for 15,000-lb single-axle load at gage point 9, Nov.-Dec. 1961.

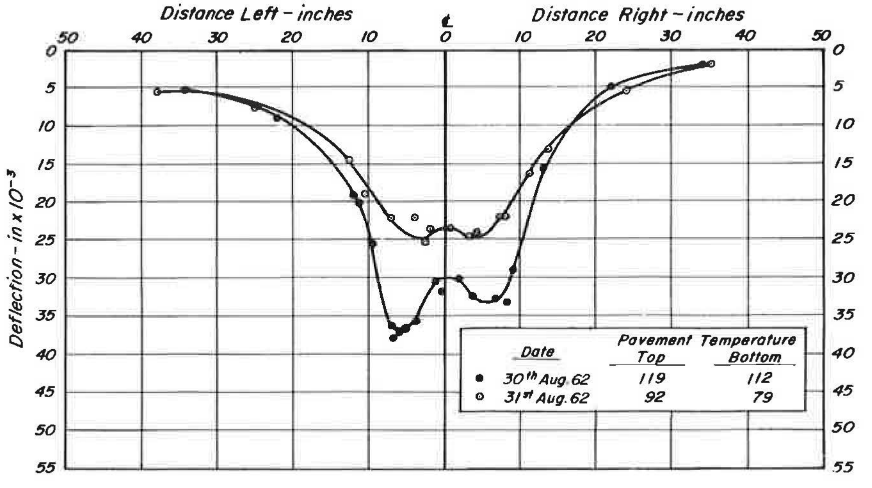

Figure 25. Transverse deflection profiles at gage point 18 illustrating the effect of temperature—15,000-lb single-axle load.

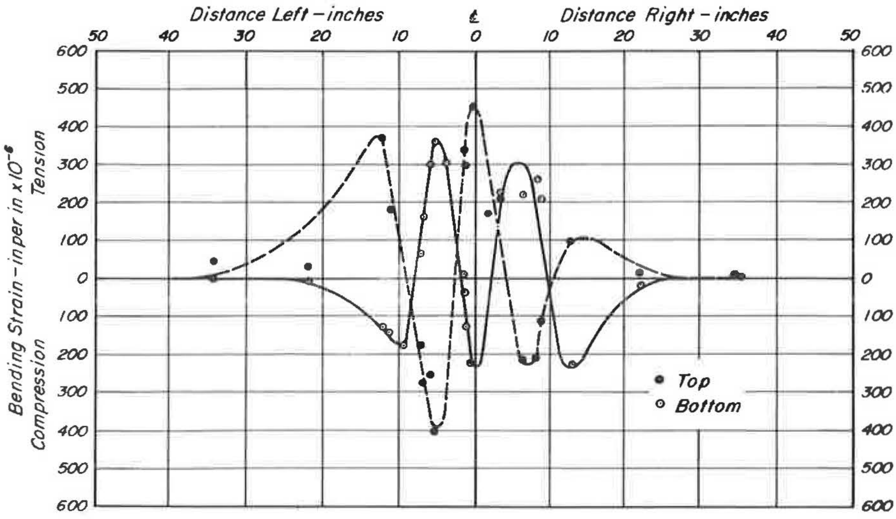

Figure 26. Comparison of transverse profiles of top and bottom transverse strain at gage point 18—15,000-lb single-axle load, 30 Aug. 1962.

Comparing the intensity of transverse strain at the surface, it is of at least the same order of magnitude as the tensile strain on the underside of the overlay in the longitudinal direction. The rate of change of this strain with distance is also interesting. Figure 25 emphasizes the importance of accurate measurement of the placement of the vehicle and also emphasizes why the strain data in Table 12 have been termed maximum observed values, since it is possible that higher values than those measured may have occurred.

DISCUSSION AND EVALUATION

From the data presented, we would conclude that the pavement is a well-constructed, dense, impervious overlay. Measurements of density and of water and air permeability substantiate this.

Road roughness measurements indicate little difference between northbound and southbound lanes, and, based on the level of roughness of 102 to 106 in./mi, the pavement is considered to be comparatively smooth. Skid resistance measurements made both in 1962 and 1963 indicate safe values of friction coefficient for all sections under wet, skidding conditions.

Since the riding quality of all sections is about the same and at a comparatively high level, it is difficult to judge performance on this basis. However, cracking has been observed in some sections of the overlay construction (Tables 10 and 11). While this cracking does not impair the riding qualities at this time, it is symptomatic of some undesirable condition, as yet undefined, and may be considered as a measure of performance. Thus the sections with the greatest amount of cracking could be considered to exhibit the poorest performance. On this basis the order of performance indicated by Tables 10 and 11 would be (1) Sections 4-0 and 8-0, (2) Section 8-W, (3) Section 4-W-2, and (4) Section 4-W-1.

However, it will be noted from Table 11 that a considerable part of the original pavement area in Section 4-W-1 (approximately 80 percent) and 4-W-2 (approximately 48 percent) was cracked. Thus, particularly in the case of 4-W-1, the cracking in the overlay probably was markedly influenced by the cracking in the original pavement. In addition (from Figs. 15 and 16), the deflections after the overlay in Section 4-W-1 are higher than the other four; thus, it cannot be compared with these sections.

Considering data in Table 11 and Figures 15 and 16 and excluding Section 4-W-1, a modified order of ranking for performance to date would be (1) Section 8-0, (2) Section 4-0, (3) Section 8-W, and (4) Section 4-W-2.

Beyond rating the pavement performance to date, it is important to attempt to explain present observed behavior and to predict future performance. The deflection measurements serve as a useful tool, particularly in the light of developments such as those presented by Hveem (8), the Canadian Good Roads Association (9), and in the AASHO Road Test report (10).

Hveem (8) has suggested that the safe limiting deflection value under a 15,000-lb single-axle load ranges from 0.020 in. for a 3-in. surfacing to the order of 0.012 in. for an 8-in. surfacing. In the actual pavement, the average thickness of the resurfacing, plus the existing pavement, is 4¾ in. On this basis a safe limiting deflection of the order of 0.016 in. would be indicated for heavy traffic conditions. From Figures 15 and 16, the average deflections for March 1962 in all sections (except 4-W-1) are in the range of 0.015 in. to 0.020 in. While the safe limiting deflection value noted above has been associated with heavy traffic, Sherman (11) has presented an analysis of the WASHO data which would indicate that this value is traffic-dependent. According to the results of the WASHO Road Test (12), critical deflections ranged from 0.045 in. to 0.030 in. for warm and cold weather, respectively. The traffic index on the 238,000 applications at WASHO, as determined by the California EWL_{57} procedure, ranged from 7.2 for the 18,000-lb single-axle load to 8.5 for the 40,000-lb tandem-axle load. According to the traffic index as of June 1964 for this project, the value was 6.9 for the southbound lane and 6.3 for the northbound lane. Since both the deflection values and traffic indexes are less than those for WASHO, no cracking should be indicated. This is the situation in the 4-0 and 8-0 sections.

Another method for evaluation of present and future performance is that developed by the Canadian Good Roads Association. According to the CGRA (9), pavement performance can be related to deflection, age and traffic. Deflections are analyzed not as average values but as an average plus 2 standard deviations ($\bar{x} + 2\sigma$). This technique recognizes not only the order of magnitude of deflection measurements but also the range intensity of the distribution of measurements. According to their analysis, a pavement whose deflection factor ($\bar{x} + 2\sigma$) does not exceed 0.025 in. should provide "good" performance for heavy traffic up to 14 yr. A pavement with a deflection factor of 0.050 in. should provide the same level of performance for about 6 yr, and a pavement with a deflection factor of 0.075 in. for about 2 yr. The average deflection data, together with standard deviations, for the various periods are summarized in Table 13. Table 14 summarizes the deflection data for each Shell Avenue section according to the CGRA method of evaluation. From this intepretation, Sections 4-W-1 and 4-W-2 would not be expected to maintain a high level of performance for longer than 3 to 5 yr. Sections 8-0 and 8-W should last about 8 yr and Section 4-0 about 10 yr.

Performance by the CGRA criteria was an overall subjective rating by a panel of 5 raters and, as such, is not directly comparable to the ratings used on the Shell Avenue Test Road. Also, environmental differences exist which would tend to make Canadian criteria conservative for conditions in California. Nevertheless, it does provide a means for comparing the efficiency of deflection testing for predicting performance.

One of the most comprehensive programs of field studies to relate deflection to pavement performance was the AASHO Road Test conducted from 1956 to 1960 in Ottawa, Ill. (10). Part of this research included a study of the deflection-performance relationship. It is pertinent here, as with the CGRA investigation, to point out that performance criteria were not based exclusively on cracking. In fact, examination of the formula used to predict performance on the AASHO project might lead to the conclusion that cracking played only a minor role in performance. However, a closer analysis will show that cracking may have a significant effect on the longitudinal profile as measured with the special Road Test profilometer and, therefore, cracking would influence this measurement, which is the prime factor in the performance term.

The general equation found from the Road Test data (10, p. 110) resulted in the following relationship for associating deflection with performance:

$$\log W_{2.5} = 7.98 + 1.72 \log L - 3.07 \log d$$

where

TABLE 13

SUMMARY OF DEFLECTION DATA

Section	Wheelpath	Date	Southbound Avg	Southbound σ	Northbound Avg	Northbound σ
4-W-1	Inner	9-61	56.58	19.86	47.22	15.21
		3-62	34.22	12.91	25.20	11.78
		3-63	35.32	13.23	25.24	3.21
		Avg	42.04	15.09	32.55	10.07
	Outer	9-61	55.74	15.30	49.05	15.75
		3-62	25.94	11.63	27.60	13.09
		3.63	29.26	8.48	32.94	12.93
		Avg	36.98	11.80	36.53	13.92
4-W-2	Inner	9-61	23.11	10.67	21.50	8.81
		3-62	19.06	8.66	18.08	6.79
		3-63	18.55	8.16	16.06	5.79
		Avg	20.24	9.16	18.55	7.13
	Outer	9-61	30.85	6.83	30.22	9.32
		3-62	20.65	5.45	21.50	6.14
		3-63	20.08	5.79	22.30	6.71
		Avg	23.86	6.02	24.67	7.39
4-0	Inner	9-61	13.48	6.28	16.75	7.00
		3-62	11.65	4.79	14.00	5.55
		3-63	11.70	4.55	10.19	4.01
		Avg	12.28	5.21	13.65	5.52
	Outer	9-61	25.26	7.79	24.68	9.19
		3-62	20.62	6.10	16.39	5.91
		3-63	18.19	6.23	15.15	4.76
		Avg	21.36	6.71	18.74	6.62
8-W	Inner	9-61	13.51	8.39	17.67	6.47
		3-62	13.58	7.11	14.41	5.10
		3-63	12.92	5.49	11.92	3.55
		Avg	13.34	7.00	14.67	5.04
	Outer	9-61	27.00	5.72	30.72	9.12
		3-62	20.18	4.73	21.79	5.23
		3-63	21.64	4.50	21.87	6.40
		Avg	22.94	4.98	24.79	6.92
8-0	Inner	9-61	19.82	6.17	20.26	5.37
		3-62	20.11	5.17	16.49	3.26
		3-63	20.55	5.17	14.71	4.27
		Avg	20.16	5.50	17.15	4.30
	Outer	9-61	29.95	7.01	27.77	5.17
		3-62	27.47	4.96	23.88	4.83
		3-63	24.63	5.30	21.62	5.43
		Avg	27.35	5.76	24.42	5.14

TABLE 14

SUMMARY OF DEFLECTION DATA BY CGRA METHOD

Section	Deflection in Southbound Lane (in. × 10^{-3}) Avg	2σ	Avg + 2σ	Adjusted to[1] 18,000-Lb Single-Axle Load
4-W-1	30	12	54	65
4-W-2	20	6	32	38
4-0	15	5	25	30
8-W	17	6	29	35
8-0	23	5	33	40

[1] Multiply by $\frac{18}{15}$ (Avg + 2σ).

$W_{2.5}$ = number of applications of axle L sustained by the pavement at the time serviceability was at level 2.5;

L = single-axle load in kips; and

d = normal fall deflection in 0.001 in., measured under a wheel load equal to L/2.

According to this equation, it should be possible to estimate the number of 10-kip single-axle load repetitions to a serviceability index of 2.5. Utilizing actual traffic and deflections on the Shell Avenue Test Road in the equation would then provide a tie to the Road Test data. Based on the results in Table 13 and the above equation, Table 15

TABLE 15

SUMMARY OF ESTIMATED ALLOWABLE TRAFFIC FOR VARIOUS SECTIONS ACCORDING TO AASHO ROAD TEST EQUATION

Section	Lane	Deflection (in. × 10^{-3})	Total No. 15,000-Lb Axle Loads	EWL_{57} × 1000	TI_{57}^{1}	EWL_{57} × 1000	TI_{63}
4-W-1	SB	30	298,000	2,235	6.7	1,640	7.0
	NB	26	445,000	3,340	7.1	2,450	7.5
4-W-2	SB	20	1,000,000	7,500	7.7	5,500	8.3
	NB	20	1,000,000	7,500	7.7	5,500	8.3
4-0	SB	16	1,990,000	14,980	8.3	10,940	8.9
	NB	15	2,450,000	18,400	8.5	13,480	9.1
8-W	SB	17	1,655,000	12,400	8.1	9,120	8.8
	NB	18	1,410,000	10,580	8.0	7,760	8.7
8-0	SB	24	574,000	4,300	7.3	3,160	7.7
	NB	20	1,000,000	7,500	7.7	5,500	8.3

[1] Traffic Index: $TI_{57} = 1.35\ (EWL_{57})^{0.11}$; $TI_{63} = 6.7\ \frac{(EWL_{63})^{0.119}}{10^6}$

summarizes the estimated number of 15,000-lb axle loads and the corresponding traffic index. Using average deflection values from the Shell Avenue Test Road, it is possible to estimate the EWL associated with the critical serviceability index of 2.5. By comparing the traffic indexes in Table 15 with those reported for the Shell Avenue project, it can be concluded that all the various test sections should be at a relatively high level of serviceability through June 1964, and this is the case.

The various analyses which utilized traffic and deflection data indicate that all sections should be performing at a high level of serviceability and, generally, that there should be no cracking.

This conclusion is also substantiated by the results of the strain measurements. When the average pavement temperature is 75 F or less, the maximum observed tensile strain does not exceed 150 × 10^{-6} in./in. (Table 12). Further, if we assume that the constant strain amplitude fatigue tests are representative of field performance, this level of strain corresponds to more than 1,000,000 load applications for 40 F and 75 F data. According to California EWL_{63} procedure, this corresponds to a traffic index of greater than 8.0, since the level of strain was associated with a 15,000-lb axle load. Thus, the strain data indicate, at least in a qualitative way, the same trends shown by the deflection data. Moreover, the observed strain data do not show any major differences between the various sections when comparisons are made at the same temperature.

Thus, on the basis of the deflection and strain data, one cannot find an explanation in terms of load application for the development of cracking in the 8-W and 4-W-2 sections and essentially no cracking in the 4-0 and 8-0 sections. However, part of the cracking in the 4-W-2 section might be related to preconstruction cracking. In spite of this, one must conclude that the cracking observed in the 8-W and 4-W sections is not completely load-associated, and some other factors may be contributing. No data are available at present to indicate what these factors may be.

Finally, it should again be emphasized that the cracking which has developed does not detract from the riding qualities of the pavement; that is, the present serviceability of all sections is high.

SUMMARY AND CONCLUSIONS

On the basis of the data presented, particularly those relating to deflection, strain and cracking, a few general conclusions are presented.

1. The deflection and strain data in themselves appear to offer no explanation for the cracking observed in sections 4-W-2 and 8-W.
2. The addition of asbestos appears to offer no advantage for this project, particularly when viewed in the light of appreciable differences in costs of mixes with and without asbestos. (Appendix B of this report includes an estimate of complete mix costs.)

It should be emphasized, however, that these conclusions apply only to the materials and specific combinations of asphalt, aggregate and asbestos used in this project, and only to the traffic and environment to which the pavement sections were subjected during the period of test.

ACKNOWLEDGMENTS

The Committee wishes to acknowledge gratefully the assistance of all those who participated in this project. Particular recognition should be given to the contributions of: F. N. Finn, Materials Research and Development, for analysis of pavement deflection measurements; T. W. Pickrell, ITTE, for electronic field measurements; and G. B. Dierking, ITTE, for preparation of figures.

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Appendix A

SHELL AVENUE TEST ROAD COMMITTEE MEMBERSHIP AND FUNCTIONS

As a result of informal discussions regarding the desirability and feasibility of constructing a test road for the purpose of conducting full-size, in-service comparative performance studies of asphalt-concrete mixtures with and without asbestos filler, a planning committee was organized from representatives of agencies interested in participating in such a study. This committee held numerous meetings to plan the experiment, supervise the construction, and to gather and analyze the numerous data obtained.

It is important to note that the size and scope of the investigative program developed by this cooperative effort would have been beyond the reasonable capabilities of any one of the individual participating organizations. Any success that this project may have had is directly attributable to the combined efforts of the individual committee members.

The members of the Committee are:

1. W. A. Garrison, Materials Engineer, Contra Costa County (Committee Chairman);
2. W. J. Kari, Technical Supervisor, American Bitumuls and Asphalt Company, Emeryville;
3. R. S. Latchaw, Construction Engineer, Contra Costa County;
4. J. A. Lettier, Products Application Engineer, Shell Oil Company, San Francisco (now D. F. Fink)
5. Vaughn Marker, Managing Engineer, Pacific Coast Division, The Asphalt Institute, Berkeley;
6. C. L. Monismith, Associate Professor of Civil Engineering, University of California, Berkeley;
7. C. J. Van Til, Staff Engineer, Pacific Coast Division, The Asphalt Institute, Berkeley (Committee Secretary);
8. C. W. Weitzel, Special Representative, Asbestos Fiber Division, Canadian Johns-Manville, Ltd., San Francisco (now Los Angeles);
9. Lew Wulff, Materials Engineer, District IV, California State Division of Highways, San Francisco.

Appendix B

ESTIMATED COMPARATIVE COSTS OF ASPHALT CONCRETE
WITH AND WITHOUT ASBESTOS FILLER

A. Without Filler

¾-in. max, with 85-100 or 40-50 penetration asphalt	$4.75 per ton

B. With Asbestos Filler

1. Base price	$4.75 per ton
2. Additional asphalt—20 lb at $25.00 per ton	0.25
3. Asbestos—50 lb at $70.00 per ton	1.75
4. Add asbestos to mix at plant	0.15
5. Additional mixing time	0.20
Total	$7.10 per ton
	$7.10
	- 4.75
Additional Cost	$2.35 per ton
Additional Cost	49.5 percent

Discussion

J. H. KIETZMAN and J. W. AXELSON, Johns-Manville Research Center, Manville, New Jersey—One of the items of interest which was not included in this report is the

TABLE 16

DATA ON CORES TAKEN FROM SHELL AVENUE IN 1961
(Chicago Testing Laboratory Report No. 07705-12, Nov. 7, 1961)

Section	Identif.	Asphalt Content (%)	Properties of Recovered Asphalt: Penetration, 77 F (100/5)	Properties of Recovered Asphalt: Ductility, 77 F (5/60)
4-0	E-1, east	5.7	36	100+
4-0	W-2, west	4.6	35	100+
Avg.		5.2	36	
4-W	E-1, east	5.8	37	100
4-W	W-1, west	6.0	30	100+
Avg.		5.9	34	
8-0	E-3, east	5.0	51	100+
8-0	W-2, west	5.0	49	100+
Avg.		5.0	50	
8-W	E-1, east	5.3	70	100+
8-W	W-1, west	5.2	57	100+
Avg.		5.3	64	

TABLE 17

DATA ON CORES TAKEN FROM SHELL AVENUE IN 1964
(Chicago Testing Laboratory Report No. 20184, Nov. 11, 1964)

No.	Core No.	Air Voids (%)	Asphalt Content (%)	Asbestos Present	Properties of Recovered Asphalt: Penetration, 77 F	Ductility 77 F	Ductility 45 F
I	1320	4.4	6.2	yes	12	11	0.0
I	1321	4.0	6.2	yes	16	23	0.0
I	1325	5.1	5.4	yes	12	12	0.0
I	1326	5.1	5.7	yes	13	16	0.3
II	1328	5.0	5.2	no	16	25	0.3
II	1329	6.7	5.7	yes	13	12	0.5
II	1331	4.7	5.7	yes	17	24	0.3
III	1323	3.5	5.2	no	15	22	0.3
III	1324	3.9	4.9	no	18	32	0.0
III	1330	4.3	5.2	no	18	41	0.3
IV	1322	4.7	5.9	yes	27	72	5.5
V	1327	4.7	5.6	yes	27	44	0.5

properties of the recovered asphalt. Cores were taken in 1961 and again in 1964 and were tested by the Chicago Testing Laboratory for aggregate gradation, amount of bitumen and properties of the recovered asphalt. Pertinent data are given in Tables 16 and 17.

Table 16 indicates that the asphalt content in all sections was considerably below the design values. The values for sections 4-0 and 8-0 check with those for the plant mixes as given in Table 9 of the report but the core values for sections 4-W and 8-W are appreciably below the plant mix values. Unfortunately, the cores in Table 17 have not been identified by section and it is not possible to check these asphalt contents. Table 16 also shows that there was not any abnormal hardening of the asphalt during the mixing and placing although the penetrations on the asphalts from the section 8 cores was somewhat lower than normally expected. To quote the CTL report "the properties of the recovered asphalts are all satisfactory for the respective penetration grades which were used in this project."

Table 17, however, shows that there has been excessive hardening of the asphalt from all sections in the three years between corings. This hardening is considerably greater than we have ever experienced in our work in the east and from all correlations that have been made between ductility or penetration of recovered asphalt and excessive cracking, it would be expected that all sections would show degeneration through cracking. In order to understand these data more fully, it is requested that the Committee supply core identification by section and give a possible explanation for the excessive hardening that took place.

The addendum is a duplicate of an inspection report made on February 21, 1963. In general, this report agrees reasonably well with the other inspection reports given. However, it is noted that we observed some 20 ft of alligator cracking in the loaded lane of the 4-0 section whereas none was reported by the Committee. Presumably, this cracking was in the 100-ft transition zone between sections.

It is hoped that this additional information will be of value in further analyses of this test road and that the study will continue. It would be desirable to know what the effect of the present cracking will be on future serviceability and whether or not cracking becomes more extensive throughout the entire project as would be expected with the low asphalt ductilities now present.

Addendum

Inspection Report*
Appearance of Test Pavement on
Shell Avenue, Martinez, California (Contra Costa County)
February 21, 1963

I. Section 8-0 Standard Mix with 85-100 Pen. Asphalt

Both loaded and unloaded lanes appear free of cracks.

II. Section 8-W $2\frac{1}{2}$ Percent Asbestos, 85-100 Pen. Asphalt

A. Unloaded lane

1. Inner wheelpath—incipient alligator cracking at 2 locations, totaling about 25 ft.
2. Outer wheelpath—one longitudinal crack, 2-ft length. Generally good condition structurally.

B. Loaded lane

Center crack attributed to paver.

1. Inner wheelpath—intermittent alligator cracking evident at three locations for a total length of about 50 ft. One longitudinal crack near LVDT gage No. 15.
2. Outer wheelpath—generally good appearance.

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*Inspection by J. H. Kietzman with C. W. Weitzel and W. A. Garrison, Materials Engineer of the Contra Costa Department of Public Works.

III. Section 4-0 Standard Mix with 40-50 Pen. Asphalt

Few intermittent longitudinal joint cracks.

A. Unloaded lane

Very good condition. No cracking evident.

B. Loaded lane

Alligator cracking 20-ft total length starting 10 ft inside transition zone at south end. Center crack intermittent but extensive, attributed to paver.

IV. Section 4-W Asbestos Mix with 40-50 Pen. Asphalt

Intermittent longitudinal joint cracks.

A. Loaded lane

Center cracking (due to paver) continuous.
1. Inner wheelpath—Alligator cracks at a few locations near middle of section.
2. Outer wheelpath—generally good.

B. Loaded lane

1. Inner wheelpath—"wet" spots 25-ft length starting at south end. Few longitudinal cracks and intermittent alligator cracking for 50-ft length.
2. Outer wheelpath—generally good appearance.

V. The 5th section with 40-50 pen. asphalt and asbestos was not officially part of the test because of the extremely poor condition of the old pavement and base. Just about every type of cracking is evident in this section.

General Comments

1. Surface texture of all of the standard mixes was tight, but considerably more open than the asbestos section. Extensive but very slight surface checking is evident, apparently still remaining from placement. Wet spots on the surface were observed with the alligator cracking and in the wheelpaths at places where cracking appears to be just beginning. The impression is that cracking is starting at the bottom of the resurfacing layer and working its way upward.

2. The inferior appearance of the asbestos mix with 40-50 pen. asphalt may be attributed to the deliberate location on the worst part of the old pavement.

3. To date, cracking has had negligible effect on ridability (serviceability) of the pavement.

4. The location of cracking almost exclusively in the inner wheelpaths is reportedly due to the thinness of the overlay pavement near the centerline of the road where the original pavement grade was high.

AUTHOR'S CLOSURE

The committee wishes to thank Messrs. Kietzman and Axelson for their discussion of the paper.

Core identification for the data presented in Table 17 of their discussion is given in Table 18.

A number of other points raised by Kietzman and Axelson deserve some comment.

In the inspection report listed as an Addendum to their discussion, the general comment "The inferior performance of the asbestos mix with 40-50 penetration asphalt may be attributed to the deliberate location in the worst part of the old pavement" was made. Out of context this could be somewhat misleading. In the report it was noted

TABLE 18

CORE LOCATIONS—1964 SAMPLES

Core No.	Location
1320	4-W uncracked area
1321	4-W uncracked area
1325	4-W cracked area
1326	4-W uncracked area
1328	8-0 uncracked area
1329	4-W cracked area
1331	4-W uncracked area
1323	4-0 uncracked area
1324	4-0 uncracked area
1330	4-0 uncracked area
1322	8-W uncracked area
1327	8-W uncracked area

in considerable detail that the first 640 ft of pavement, that containing 40-50 penetration asphalt with asbestos, was not considered a part of the test since the underlying conditions were not comparable to the remaining approximately 2, 500 ft of pavement. Thus, while cracking data are reported for this section (4-W-1), no comparisons are made with the other sections.

With regard to the comment on alligator cracking in the 4-0 section, this cracking occurred in the transition section between 4-0 and 4-W and thus was not reported.

The discussors call attention to the fact that the asphalts have hardened excessively in the sections covered by the core data presented in Table 17.

Whether or not this is excessive hardening is not pertinent at this point since comparisons were made between sections and, as seen in Tables 17 and 18, this hardening is about the same in all sections. As noted in the report, the sections containing asbestos exhibit cracks while those without have no cracking. Furthermore, the cracking which appears cannot be explained in terms of the analyses presented. Thus, to present conjecture as to the cause of cracking in the asbestos sections would be difficult since there is no unanimity of opinion among the members of the committee. Some feel that the cracking may be due to load stresses, others to stresses resulting from volume change (13) and still others to a combination of load stresses and those associated with volume changes.

In conclusion it should be noted that observations, though not as extensive as those made during the first 3 years, will be continued on the project through at least 5 years. Thus, some measure of the influence of existing cracking on future serviceability will be obtained.

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