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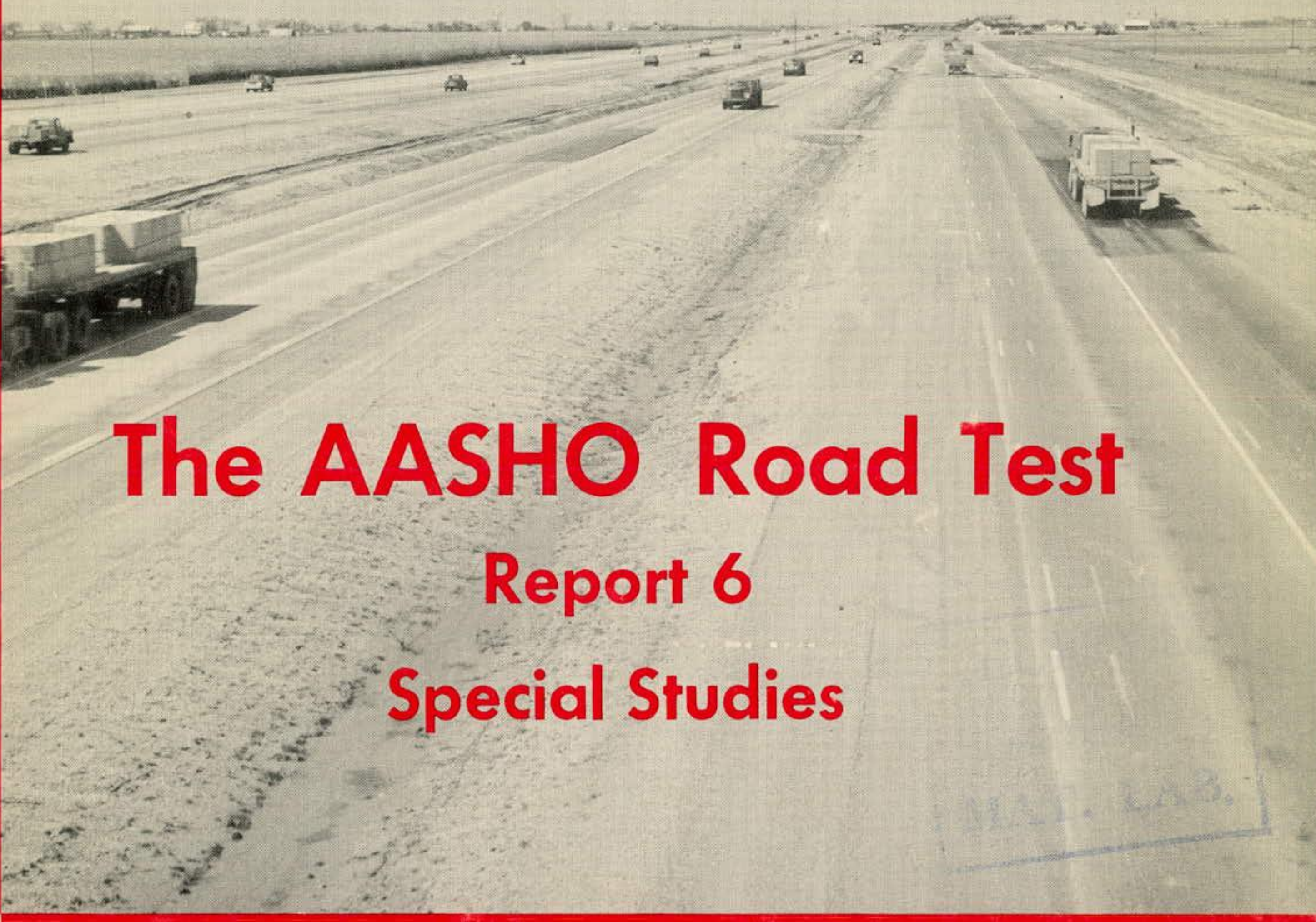
HIGHWAY RESEARCH BOARD
Special Report 61F



The AASHO Road Test
Report 6
Special Studies

National Academy of Sciences—
National Research Council

publication 955

HIGHWAY RESEARCH BOARD

Officers and Members of the Executive Committee

1962

The AASHO Road Test

Report 6

Special Studies

By the

HIGHWAY RESEARCH BOARD

of the

NAS-NRC Division of Engineering and Industrial Research

Special Report 61F

Publication No. 955

National Academy of Sciences—National Research Council

Washington, D.C.

1962

This is one of a series of reports of work done under a fiscal agreement of June 10, 1955, between the National Academy of Sciences and the Bureau of Public Roads relating to AASHO Road Test Project; and under individual agreements covering Cooperative Highway Research Project (AASHO Road Test) made between the National Academy of Sciences and the several participating state highway departments, members of the American Association of State Highway Officials.

Included in the series are the following reports:

Report	*Subject*	*HRB Special Report No.*
1	History and Description of Project	61A
2	Materials and Construction	61B
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4	Bridge Research	61D
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Available from the
Highway Research Board
National Academy of Sciences—
National Research Council
Washington 25, D. C.

Library of Congress Catalog Card No. 61-60063

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Preface

The AASHO Road Test was conceived and sponsored by the American Association of State Highway Officials as a study of the performance of pavement and bridge structures of known characteristics under moving loads of known magnitude and frequency. It was administered by the Highway Research Board of the National Academy of Sciences—National Research Council, and was considerably larger and more comprehensive than any previous highway research study.

This is the sixth in a series of major reports on the AASHO Road Test. The first report is a history and description of the project; the second is a detailed account of the materials and construction of the test facilities; the third is a description of the pavement maintenance and vehicle operations; the fourth is a detailed account of the bridge structure experiment; and the fifth describes the pavement research and its analyses. AASHO Road Test Report 7 will summarize the findings and conclusions of all previous reports.

This report is presented in ten chapters covering those studies conducted during the main test which were not directly associated with the principal efforts of the pavement or bridge research branches and those studies conducted primarily for the Department of the Army during the special study program following the main test.

The experiment design and the instrumentation available did not have the refinements to detect the effects of the variables of tire pressure, tire design, vehicle suspensions, and the several vehicle and axle configurations used in the studies.

The tests were conducted primarily to detect gross trends in the dynamic measurements of strain, deflection, and dynamic axle loads, among others.

Within the limits of the test facilities, these special studies were designed to indicate trends and parameters for future research, as well as to provide a basis for better testing techniques and instrumentation to identify and measure the critical response in the vehicle and the pavement. The findings of these special studies were not definitive, but the summaries reported should suggest further areas of research.

Acknowledgments

Personnel from many organizations assisted in preparing for and carrying out the extensive post-traffic and other special studies. It is impractical to list in this report the names of all individuals who participated. However, the efforts of the following are particularly acknowledged:

The Bureau of Public Roads, U. S. Department of Commerce, for technical advice, equipment and personnel services in a great many areas.

The Department of Defense, for the services of personnel of the U. S. Army Transportation Corps Road Test Support Activity (AASHO) and for instruments, technical advice and services of personnel from the Transportation Corps, Department of the Army.

The Automobile Manufacturers Association and the Truck Trailer Manufacturers Association, and their member companies, for technical advice and services in many areas and specifically for assistance in obtaining special vehicles for the post-traffic studies.

The Tire Industry, and representatives of member companies, for technical advice and services, specifically in regard to special tires used in the post-traffic special studies.

The General Motors Corporation, for the use of the skid resistance equipment and trained personnel in conducting the several series of measurements.

The Caterpillar Tractor Company, the Hutchens and Son Metal Products Company, the International Harvester Company, and the White Motor Company, for the use of special vehicles in the post-traffic special studies.

The Corps of Engineers, Department of the Army, for cooperation in the study of two special testing procedures.

The Ordnance Corps, Department of the Army, for the loan of equipment for use in these studies.

The Shell Oil Company, for the use of their road vibration machine and analyses of the data from an extensive testing program.

The Illinois Division of Highways, for technical personnel in the field operation and analyses of the post-traffic studies.

The following organizations for the services of resident observer consultants: The Asphalt Institute, the Portland Cement Association, the American Trucking Associations, the Canadian Good Roads Association, the Department of Highways of the Province of Ontario, Canada, and the German Highway Research Board.

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THE AASHO ROAD TEST

Report 6

Special Studies

Chapter 1

Description of the Project

This chapter is a brief description of the AASHO Road Test including its background and concepts, the location and layout of the test facilities, traffic operation, pavement maintenance, and environmental conditions.

1.1 BACKGROUND AND CONCEPTS

The AASHO Road Test was conceived and sponsored by the American Association of State Highway Officials as a study of the performance and capabilities of highway pavement and bridge structures of known characteristics under moving loads of known magnitude and frequency. The test was intended to develop engineering knowledge which could be used in the design and construction of new highway pavements and bridges, and in the preservation and improvement of existing pavements. It was intended also that the findings be used, in conjunction with data from other research, in the advance toward an ultimate goal of determining an optimum economic balance between vehicle operating costs and the costs of highways.

The project was financed by 49 states, the District of Columbia, the Commonwealth of Puerto Rico, the Bureau of Public Roads of the U. S. Department of Commerce, the Automobile Manufacturers Association, the American Petroleum Institute, and the American Institute of Steel Construction. The Department of Defense, through its Army Transportation Corps Road Test Support Activity, furnished drivers for the test vehicles and personnel for supervision of the drivers. Foreign countries and domestic materials and transportation associations furnished resident observers and staff consultants.

The basic concepts of the AASHO Road Test were outlined in 1952 by the Working Committee of the AASHO Committee on Highway Transport. This committee also selected the test site near Ottawa, Ill., about 80 mi southwest of Chicago (Fig. 1).

In November 1954, the American Association of State Highway Officials approved construction of the test facilities. In February 1955, the Highway Research Board, with the approval of the National Academy of Sciences—National Research Council agreed that the Board would administer and direct the project.

A detailed history and description of the project is given in AASHO Road Test Report 1 (HRB Special Report 61A), and Report 2 (HRB Special Report 61B) is a comprehensive account of the materials and construction of the test facilities. Report 3 (HRB Special Report 61C) describes the traffic operation and pavement maintenance. Report 4 (HRB Special Report 61D) is a complete account of the bridge research, and Report 5 (HRB Special Report 61E) describes the pavement research and its analyses.

Basic data from all Road Test experiments are filed on IBM cards and in other forms in numbered data systems. Data systems associated with bridge research are listed in Appendix A, Road Test Report 4, and all other data systems are listed in Appendix I, Road Test Report 5.

The specific objectives of the project placed major emphasis on the determination of significant relationships between the performance of pavements of various designs and the loading applied to them, on developing means of evaluating pavement capabilities, and on determining the significant effects of loading on bridges of known design and characteristics.

The objectives directed the project staff to provide a record of the type and extent of effort and materials required to maintain each of the pavement test sections, or portions thereof, in

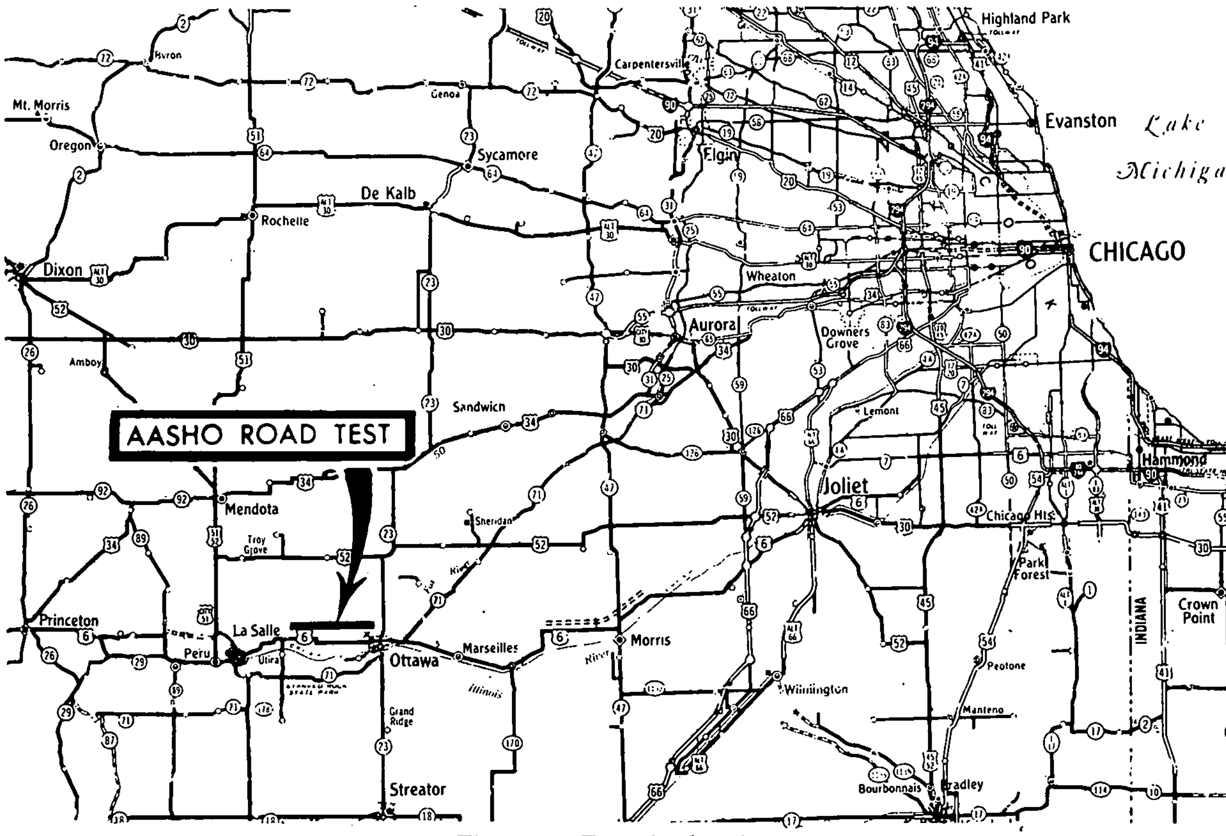


Figure 1. Test site location.

a satisfactory condition until discontinued for test purposes. The staff also was directed to conduct special studies dealing with such subjects as paved shoulders, base types, tire size and pressure, and heavy military vehicles with the aim of correlating these studies with the results of the basic research.

The special studies dealing with tire size and pressures, heavy military vehicles and other associated research are detailed in this report.

Formal agreements regarding special studies were concluded among the Secretary of the Army, the President of the American Association of State Highway Officials and the Executive Officer, National Academy of Sciences—National Research Council.

In a letter of June, 1956 from the Secretary of the Army to the President of AASHO, it was stated:

> It is understood that . . . Special Studies of particular interest to the Department of Defense will be conducted on a cooperative basis during and after the regular test, and that the National Academy of Sciences, Highway Research Board Road Test staff will assist in the conducting of the agreed-to special studies both during and after the regular tests, make necessary analyses, and prepare appropriate reports.

In July 1956, the Executive Officer, National Academy of Sciences—National Research Council, wrote to the Secretary of the Army agreeing to the conditions set forth.

1.2 TEST FACILITIES AND TRAFFIC OPERATIONS

1.2.1 Site Location and Layout of Test Facilities

The test facilities were constructed on an 8-mi right-of-way which was on the alignment of U. S. Interstate Route 80 northwest of Ottawa, Ill.

As shown in Figure 2, the facilities were built in six loops. The four larger loops (3 through 6) were constructed for testing under tractor semitrailer type vehicles. Loop 2 used extensively in the post-traffic special studies, was constructed for testing under light truck traffic; and Loop 1 was designed for testing with static, creep speed and vibrating loads and for observations of the effects of time and weather on pavements with no traffic.

Each loop was a segment of a four-lane divided highway whose tangents were connected by turnarounds at both ends to form a

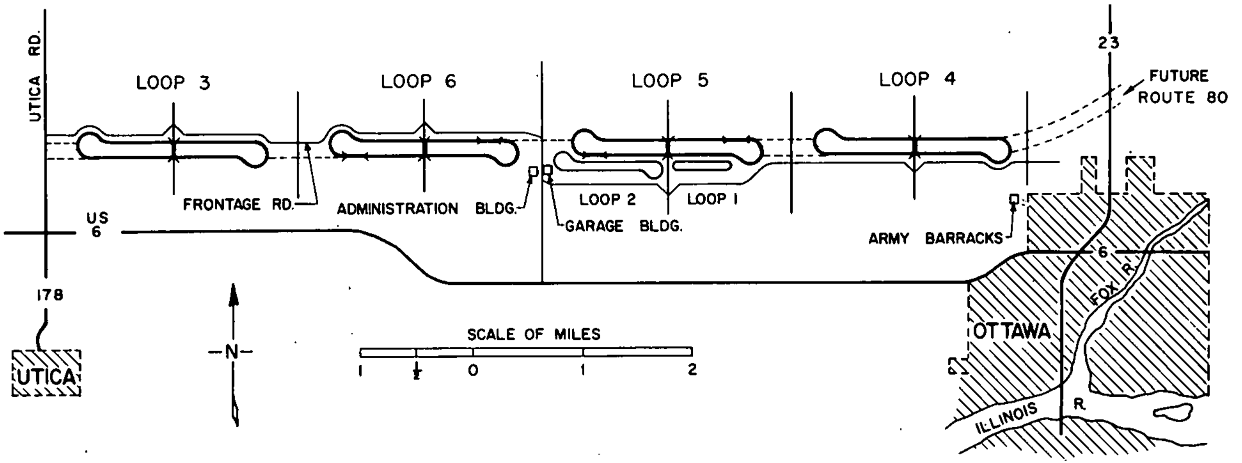


Figure 2. Map of AASHO Road Test.

two-lane loop. Tangents were 6,800 ft long in Loops 3 through 6; 4,400 ft in Loop 2; and 2,000 ft in Loop 1. The north tangent and the east turnaround of each loop was a flexible-type pavement, and the south tangent and west turnaround was a rigid-type pavement.

On each tangent the pavements were constructed in short sections of varied design. The design variables were the different levels of thickness of the component layers of materials. These combinations of sections made up a complete factorial experiment in each tangent.

Other design variables, such as paved shoulder and different base types, were incorporated in other sections in the four large loops. These were not included in the factorial experiments.

Each structural section was constructed the full 24-ft width, but because of two different traffic treatments was separated into two test sections by the centerline. Abutting sections were separated by a short transition pavement.

Including Loop 1, there were 836 pavement test sections. Of these, 716 were subjected to controlled traffic loading.

The test facilities for the bridge experiments were constructed at four locations in Loops 5 and 6. At each location four individual 50-ft span bridges were constructed on a common substructure. Each bridge was a simple span one-lane structure consisting of three beams with a reinforced concrete deck slab. In eight, the beams were wide-flange rolled steel I-sections with and without tension cover plates; in four, the beams were precast, prestressed concrete sections; in four, the beams were reinforced concrete T-beams cast monolithically with the slab.

The bridge designs were based on stress levels substantially greater than those used in current practice.

A limited amount of data was accumulated during the post-traffic special studies to determine the effect of various loadings and axle arrangements on the bridges remaining in test.

A comprehensive description of the test site and facilities is included in Road Test Report 1 (HRB Special Report 61A).

1.2.2 Traffic Operations

The layout of the test loops provided 10 lanes for traffic operation. Ten different axle load-

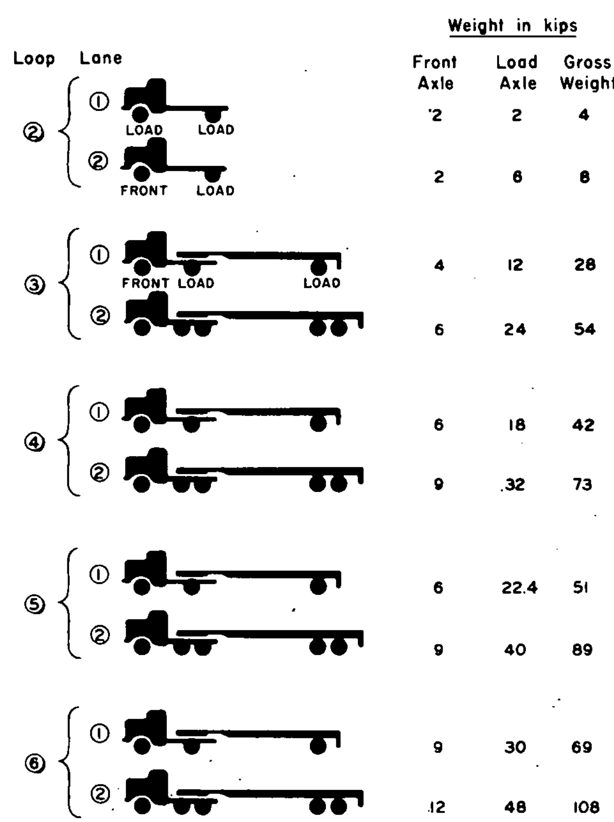


Figure 3. Axle load and vehicle type by lane.

axle arrangement combinations were selected, one for each lane. Figure 3 shows the assignment of axle loads and vehicle types for each traffic lane.

Full-scale test traffic began on November 5, 1958. At that time, the project had a fleet of 70 vehicles for a planned operation of six vehicles per lane on Loops 3 through 6, four vehicles on the inner lane of Loop 2, and eight vehicles on the outer lane of Loop 2. These vehicle assignments allowed the same rate of loaded axle applications on each lane. The original operations schedule called for approximately 19 hours per day, 6 days per week.

The rate of axle applications was materially increased in January 1960 by the addition of 48 vehicles to the fleet for a planned operation of ten vehicles per lane for Loops 3 through 6, six on the inner lane and twelve on the outer lane of Loop 2. The weekly operation schedule was increased to 7 days at this time and continued on this basis until July 1960.

Test traffic was operated until November 30, 1960, at which time approximately 1,114,000 axle load applications had been recorded. A detailed description of the traffic operation is included in Report 3 (HRB Special Report 61C).

The pavement test sections still in service after the regular test traffic were allowed to remain idle from November 1960 until February 1961, when frost had left the pavement structure. At that time the lanes to be used during the special studies program were subjected to a limited number of light axle applications. Table 1 is a record of this conditioning traffic.

TABLE 1

CONDITIONING TRAFFIC

Loop	Lane	Axle Load (kips)	Applications
2	1	1.0[1]	2,453
	2	1.0	2,313
4	1	9.0	2,474
6	1	15.0	2,470
	2	24.0	2,484

[1] Loop 2 pickups, no load.

1.2.3 Pavement Maintenance

One of the primary special studies involved the operation of 32-kip tandem axle loads on Loop 2 (see Chapter 2). These loads were greatly in excess of the 2- and 6-kip single axle loads operated on the loop during the period of regular test traffic. Therefore, it was necessary to strengthen many of the light pavement sections before beginning the special study. Some of these sections had survived the regular test traffic without maintenance, but others had

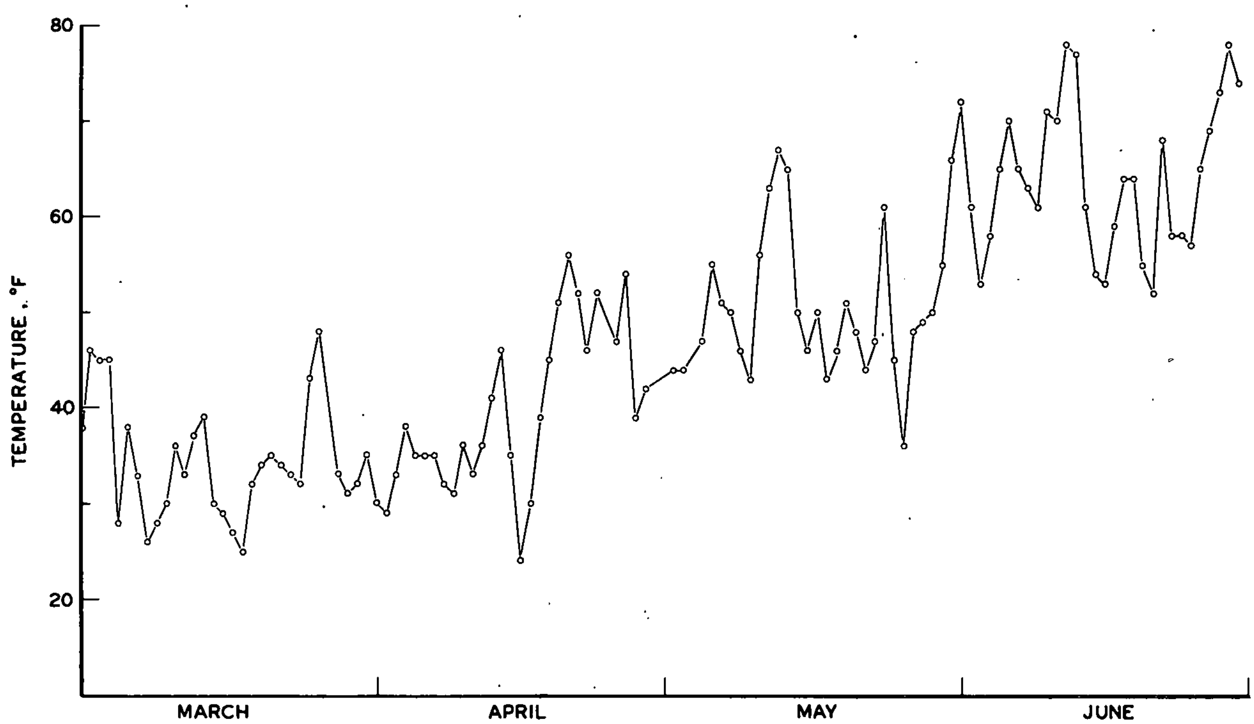


Figure 4. Daily mean air temperature, 1961.

been previously reconstructed or overlaid. These sections were considered separately in the Loop 2 performance study.

The depth of the overlay required for each of the sections was determined from the main Road Test performance equations considering the number of 32-kip axle applications scheduled. From these determinations the overlay thickness was set (to the nearest 1½ in.). Some of the sections which had been rebuilt during the main traffic test were strengthened with an overlay of 1½ in., while others received up to 4½ in.

The maintenance prior to traffic was done on a contract basis between March 15 and March 27, 1961. A total of 2,115 tons of hot-mix asphalt, furnished by a local producer, was used on the tangents and east turnaround of Loop 2. This pre-traffic maintenance program included 62 test sections.

All of the maintenance work performed during the special traffic operation period was done by the project's Maintenance Branch. Most of the repairs were made using hot-mix asphaltic concrete for overlay. A total of 1,530 tons of material was used in these operations.

As far as possible, the maintenance work performed by the project forces was done dur-

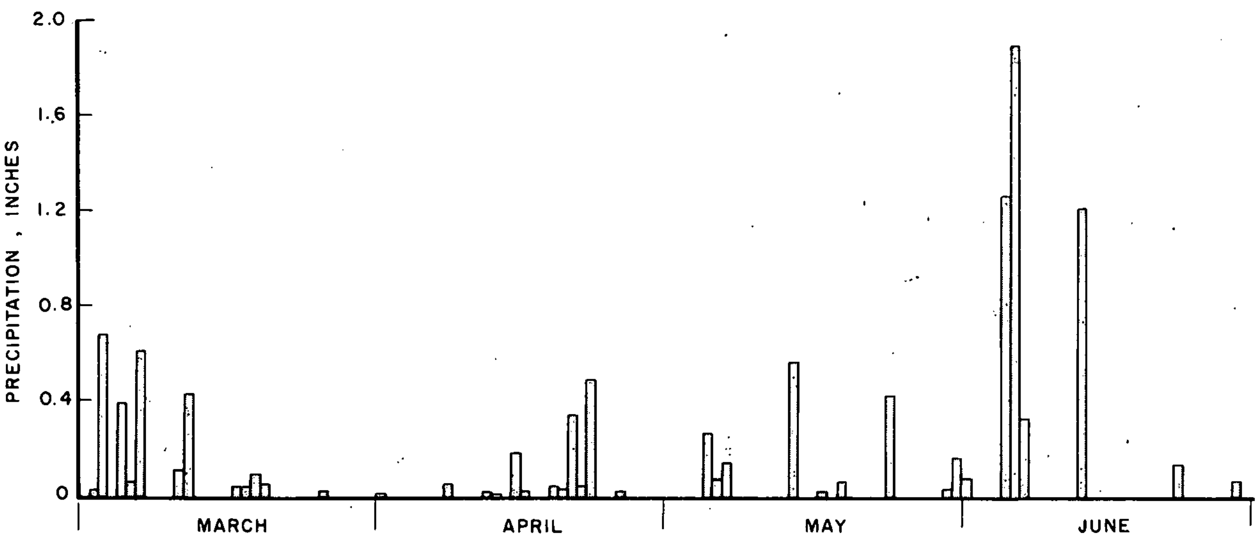


Figure 5. Daily precipitation, 1961.

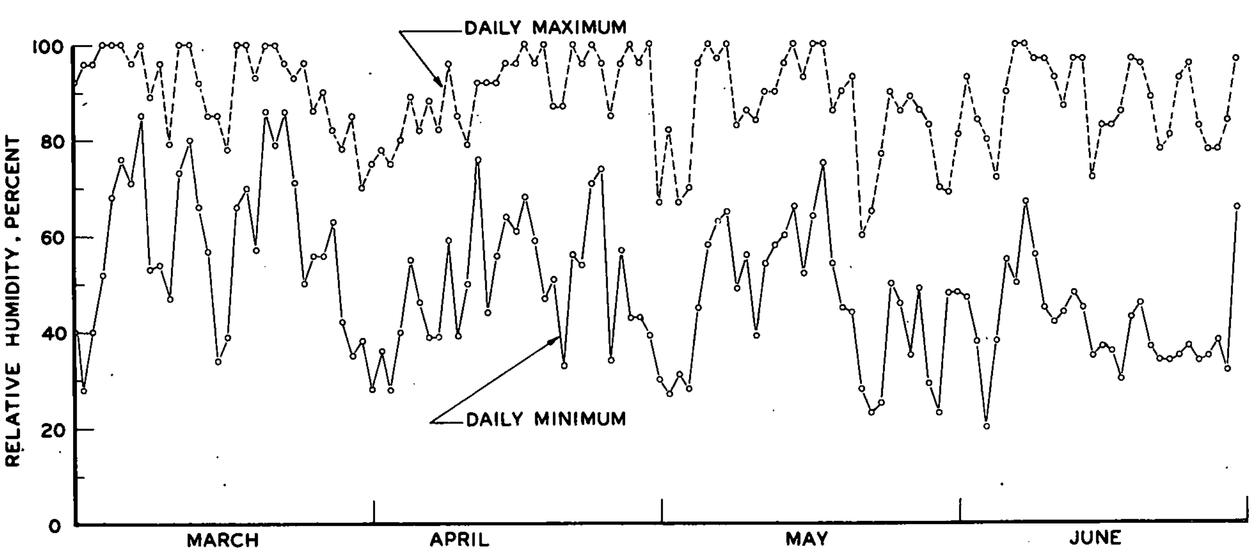


Figure 6. Relative humidity, Peoria, Ill., 1961.

TABLE 2

PAVEMENT MAINTENANCE RECORDS [1]

Original Section Design	Main Road Test Maintenance	Overlay Thickness Placed (in.)	Traffic Maintenance Overlay (in.)
2–3–4		3	
2–3–4		3	
2–6–4			5
2–6–4			4.5
0–6–4	1-sq ft skin patch	3	
0–6–4	2-in. overlay	1.5	
0–3–4	2-sq ft skin patch	4.5	
	Reconstruct and 1-in. overlay		
1–3–4			5
	Reconstruct and 1-in. overlay	1.5	
1–6–4		3	
	Reconstruct	3	
1–0–0	Reconstruct	3	
	Reconstruct	1.5	
0–0–4	Reconstruct and 1-in. overlay	3	
	Reconstruct and 1-in. overlay	1.5	
0–3–0	Reconstruct and 1-in. overlay	3	
	Reconstruct and 1-in. overlay	1.5	
1–0–4	Reconstruct and 1-in. overlay	3	
	Reconstruct and 1-in. overlay	1.5	
2–0–4	16-sq ft skin patch	5	1.5
	Reconstruct and 1-in. overlay	1.5	
2–3–0	10-sq ft skin patch	4.5	
	Reconstruct and 2-in. overlay	1.5	
0–0–0	Reconstruct and 1-in. overlay	3	
	Reconstruct and 1-in. overlay	1.5	
0–6–0		4.5	
	Reconstruct and 1-in. overlay	1.5	
2–6–4			4.5
			4.5
3–0–4		3	1.5
2–0–0	Reconstruct and 2-in. overlay	3	
	Reconstruct and 2-in. overlay	1.5	
3–3–0		3	
	2-in. overlay	1.5	
2–6–0			4.5
			4.5
5–3			
3.5R–3			
2.5R–0	Repair blow-up 1x12 ft	4 5	
	3.5-in. overlay	1.5	
3.5R–6			
2.5–6			4.5
		4.5	
3.5–6			4.5
			4.5
2.5R–6		4.5	— [3]
		4.5	
2.5–3		4.5	— [4]
		4.5	
3.5R–0			— [6]
3–0–4	Reconstruct	1.5	
2–3–4		3	
	2-in. overlay	1.5	
1–3–0	2-in. overlay	3	
	Reconstruct and 2-in. overlay	1.5	
3–3–4			4.5
			4.5
0–6–0		4.5	
	Reconstruct and 2-in. overlay	1.5	
3–6–0			5
			4.5
0–0–0	Reconstruct and 1-in. overlay	3	
	Reconstruct and 1-in. overlay	1.5	
0–0–4	Reconstruct and 1-in. overlay	3	
	Reconstruct and 1-in. overlay	1.5	
1–6–0			5
	2-in. overlay	1.5	
2–6–0			4.5
			5
2–3–0			5
	2-in. overlay	1.5	1.5
0–6–4			4.5
	1-sq ft resurfaced		4.5
3–6–4			
0–3–4		4.5	
	Reconstruct and 2-in. overlay	1.5	
0–3–0	Reconstruct	3	1.5
	Reconstruct and 2-in. overlay	1.5	
3–0–0		3	1.5
	Reconstruct and 2-in. overlay	1.5	
3.5R–0			4.5
5R–6			
5–3			
2.5R–3		4.5 [2]	5 [2]
	3-in. overlay	1.5	
5–0			
5–6			
2.5–0	Repair blow-up 9x12 ft	5.5	
	3-in. overlay	1.5 [2]	
5R–0			
5R–3			
3.5–3			5
			4.5
3.5–0		4.5	
		4.5	
3.5R–3		3 [5]	
		3 [5]	

[1] See Report 3, Traffic Operations and Pavement Maintenance, for definition of terms.
[2] One-half of section only.
[3] Resurfaced 50 sq ft.
[4] Resurfaced 470 sq ft.
[5] Part of section.
[6] Resurfaced 90 sq ft.

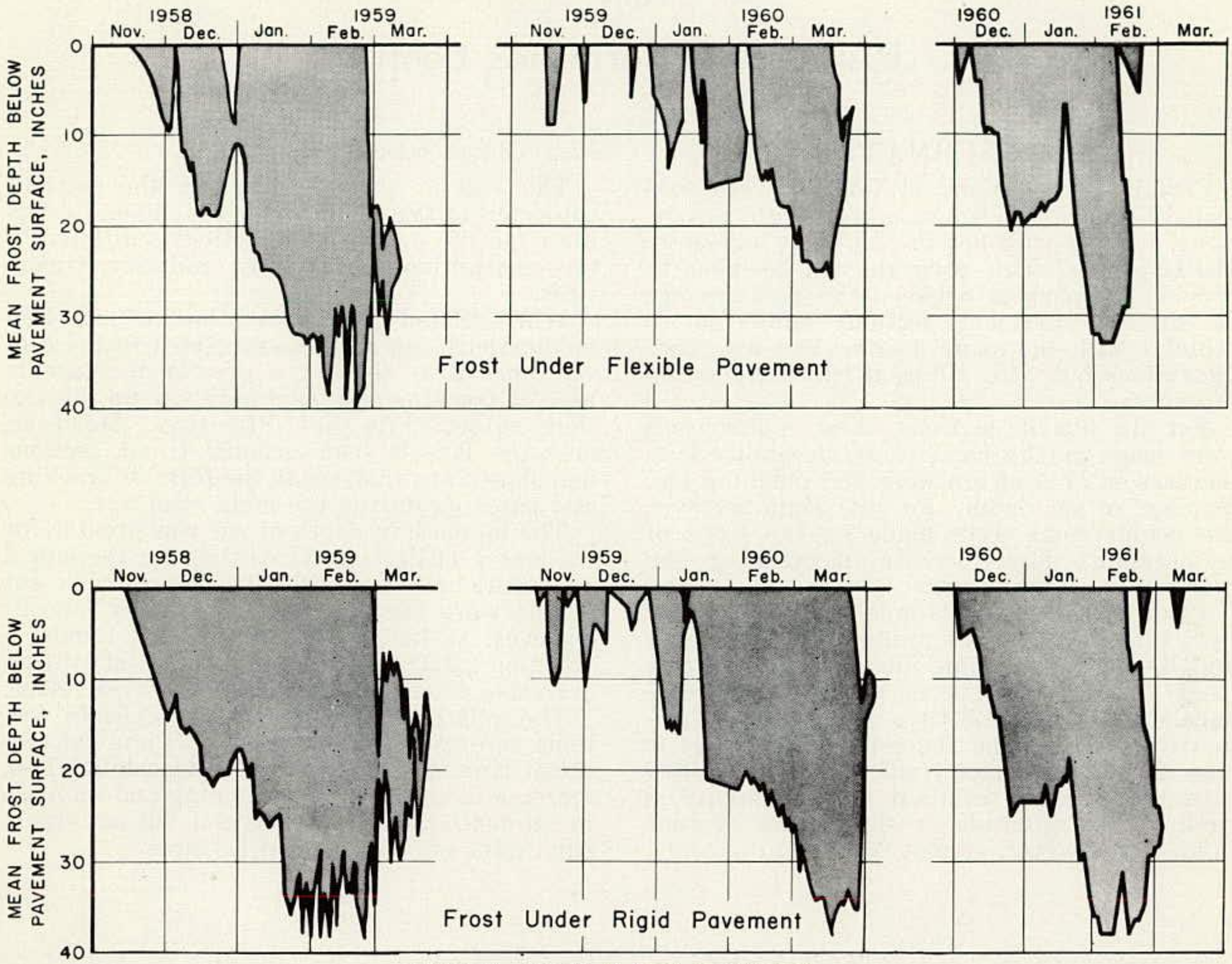


Figure 7. Depth of frost penetration.

ing the no-traffic break period each day in order to minimize delays in traffic operations.

Table 2 is a complete record of the maintenance during this special study.

1.2.4 Environmental Conditions

During the period of regular test traffic, three small weather stations were maintained. Records of air temperature, precipitation, wind velocity and frost depth were made at one or more of these locations at frequent intervals. Summaries of these data are included in Report 5 (HRB Special Report 61E).

Weather conditions preceding and during the special study program are shown in the following figures:

Figure 4 shows the daily mean air temperature for the period March through June 1961. The weather station was located at the center of the project site. The normal mean temperature of this period is 55 F.

Figure 5 shows the daily precipitation of all forms at the same location for the period March through June 1961. The total precipitation for this period was about 3.5 in. below normal for the Road Test area.

Figure 6 is a record of the maximum-minimum daily relative humidity for a station approximately 70 mi from the Road Test site.

Figure 7 is a plot of the depth of frost under both the rigid and flexible pavements for the winter immediately preceding the special studies.

In general, the weather conditions preceding and during the special study program were near normal for the area. The greatest variation from normal was in the amount of precipitation.

Chapter 2

Pavement Performance, Loop 2

2.1 SUMMARY

Certain test sections in Loop 2 were subjected to 32-kip tandem axle loads with conventional tires or with low pressure-low silhouette (LPLS) tires. This program was designed to provide comparison between the performance of similar pavement sections subjected to vehicles with the same loading and axle configurations but with different tire designs and pressures.

For the flexible sections, these comparisons were made on the basis of serviceability loss, increase in area of cracking and patching and increase in rut depth. For the rigid sections, the comparisons were made on the basis of serviceability loss, increase in cracking and increase in pumping score.

Prior to these studies lane 1 of Loop 2 had been subjected to over a million 2-kip axle loads and lane 2 to the same number of 6-kip axle loads. Since, in the special pavement performance study, the LPLS tires were operated primarily in lane 1 and the conventional tires in lane 2, the previous traffic history possibly influenced the test results in favor of the LPLS tires. The magnitude or significance of such influence, however, cannot be known.

Flexible Pavement

The loss in serviceability for the sections subjected to the LPLS tires was generally less than for the comparable sections subjected to the conventional (standard military tread) tires.

When related at a common level of axle load applications, the sections subjected to the conventional tires showed a greater increase in area of cracking and patching than did the sections subjected to the LPLS tires. However, only the lane 2 (conventional tires) sections had shown any distress in the form of cracking and patching during the main road test.

The increase in depth of rut was greater for the lane 1 (LPLS) sections than for the lane 2 (conventional) sections. The before-traffic rut depths were less for lane 1 than for lane 2; however, as indicated in Road Test Report 5 (Section 2.2.3), the rate of increase of rutting decreases after a certain depth of rut develops.

The relative performance of replicate sections subjected to both the LPLS and conventional tires on the basis of serviceability loss, increase in cracking and patching and increase in rut depth indicate a beneficial, but not highly significant effect of the LPLS tires.

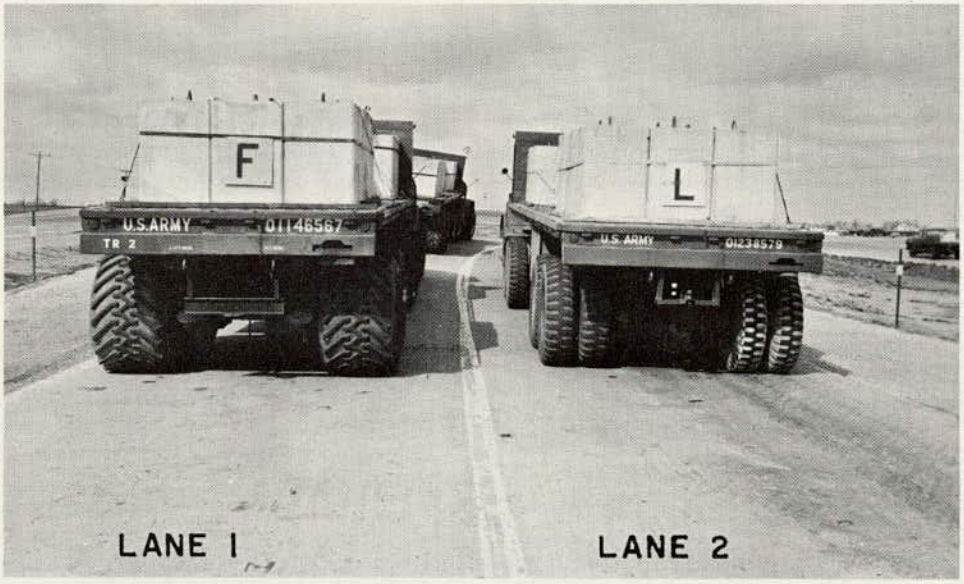


Figure 8. M-52 Tractor-semitrailers equipped with LPLS tires (lane 1) and conventional tires (lane 2).

Rigid Pavement

The loss in serviceability for the sections subjected to the LPLS tires was generally less than for the sections subjected to the conventional tires. In most comparisons, however, the loss in serviceability amounted to only a few tenths of a point.

The increase in lineal feet of cracking in lane 1 (LPLS) was greater than the increase in lane 2 (conventional). The formation of minor cracking apparently occurred under a few applications but did not develop into the class of cracking which detracts from the serviceability of the section. The lane 1 sections in the study had no previous history of crack development.

Pumping had developed only in the lane 2 sections during the main Road Test and continued to progress more rapidly during this study than pumping in the lane 1 sections. Progression of pumping, once started, was also observed during the main Road Test.

The loss in serviceability for the four replicate sections was not significant and thus, the comparison of the performance of these sections must be made on the basis of the increases in pumping score and length of cracking. In general, the difference in the increase of these two measurements was slight (see Table 16).

In the main Road Test, no significant effect of concrete reinforcement on the performance of the test pavements was found. Therefore, it is interesting to note in Figures 20 and 21 that reinforced pavements performed appreciably better than non-reinforced pavements of the same slab thickness in these special studies.

The following sections describe the experiment design, equipment used, data and findings in detail.

2.2 SCOPE

Four tractor-semitrailer units, with military designation M-52, were loaded to 32 kip on the tandem axles and operated over certain test sections in Loop 2. The sections had been previously subjected to 1,113,760 axle applications of 2-kip loads on the inner lane (lane 1) and to the same number of 6-kip loads on the outer lane (lane 2).

Two vehicles equipped with standard military tread tires at 70 psi operated in lane 2, and two equipped with the LPLS tires at 35 psi operated in lane 1 (mean operating pressures 42 and 76 psi). Figure 8 is a view of the loaded vehicles and tires, and Table 3 gives the characteristics of the vehicles. Figure 9 shows the tire contact prints for the conventional military and the off-highway low pressure-low silhouette treads. It should be noted that the LPLS tire is designed for a 6,000-lb wheel load at 35-psi inflation pressure. During this special study, each traffic lane was subjected to a total of 16,446 axle applications. Table 4 gives the

TABLE 3

CHARACTERISTICS M-52 TRACTOR-TRAILER

Vehicle Number	Axle Load (kips)	Tire Size	Tire Pressure Cold (psi)	Gross Load (lb)	Axle Load (lb)					Axle Spacing (in.)				Gage[1] (in.)	Gross Contact Area (sq in.)
					Axle 1	Axle 2	Axle 3	Axle 4	Axle 5	1–2	2–3	3–4	4–5		
M–52	32 T	46.00x24(LPLS)	35	71,180	9,640	15,050	15,090	15,280	16,120	87	54	247	54	72	204.1[2]
M–52	32 T	11.00x20(Conv)	70	73,490	9,140	16,250	15,840	15,200	17,060	87	54	247	54	72	133.4

[1] Transverse spacing, in inches, between center of dual tires or centers of LPLS tires.
[2] Actual contact area, off-highway tread is approximately 30 to 40 percent of gross contact area (see Fig. 9).

daily rate of axle applications, the delay due to vehicle and roadway maintenance, and the operating hours. Operations were scheduled on the basis of three 6-hr or two 8-hr driving shifts per day, 5 days per week, from February 7 to June 3, 1961. Figure 10 shows the actual applications as compared with those theoretically possible, showing the amount of downtime for maintenance.

The sections selected for study were those that had survived the regular test traffic in both lanes, plus three flexible and two rigid sections of the lightest designs that were available in one lane only. The latter were of interest to the Department of Defense. Figure 11 shows typical rigid and flexible pavement sections prior to the test.

Since the 24-ft wide structural sections had previously carried 6 kips in lane 2 and 2 kips in lane 1, it was recognized that the two one-lane wide test sections of a structural section were not identical at the outset of the study. In an attempt to determine the magnitude of the error thus introduced, replicate structural sections (with identical designs) were included in each tangent. The traffic pattern allowed an interchange of lanes for the vehicles at one location in each tangent. Thus, the replicated sections were subjected to the vehicles equipped with both the LPLS and conventional tires. Figure 12 shows the guide lines indicating the lane changes.

In addition to the study of original sections, an investigation of the effectiveness of an asphaltic concrete overlay of variable thicknesses on both pavement types was incorporated.

2.3 DESCRIPTION OF MEASUREMENTS

Observations and measurements were patterned on those made during the regular test

TABLE 4

SUMMARY OF VEHICLE OPERATIONS, LOOP 2

Daily Applications		Accumulated Applications		Daily Time (hr)		Accumulated Time (hr)	
Lane 1	Lane 2	Lane 1	Lane 2	Down	Operating	Down	Operating
34	38	34	38	17.4	.6	17.4	.6
304	304	338	342	7.0	11.0	24.4	11.6
156	156	494	498	13.5	4.5	37.9	16.1
360	360	854	858	0.0	10.0	37.9	26.1
108	112	962	970	3.5	2.5	41.4	28.6
228	232	1,190	1,202	3.9	7.1	45.3	35.7
396	396	1,586	1,598	0.0	12.0	45.3	47.7
398	398	1,984	1,996	0.0	12.0	45.3	59.7
304	304	2,288	2,300	2.0	10.0	47.3	69.7
244	244	2,532	2,544	0.0	6.0	47.3	75.7
92	92	2,624	2,636	13.1	2.0	60.4	77.7
200	200	2,824	2,836	10.0	5.8	70.4	83.5
136	136	2,960	2,972	12.0	3.0	82.4	87.3
484	484	3,444	3,456	0.0	12.2	82.4	99.5
230	100	3,674	3,556	2.6	5.9	85.0	105.4
358	358	4,032	3,914	0.0	10.0	85.0	115.4
504	504	4,536	4,418	0.0	16.0	85.0	131.4
460	460	4,996	4,878	4.0	12.0	89.0	143.4
216	216	5,212	5,094	2.0	13.8	91.0	157.2
520	520	5,732	5,614	3.0	13.0	94.0	170.2
502	502	6,234	6,116	0.0	15.0	94.0	185.2
596	596	6,830	6,712	0.0	16.0	94.0	201.2
454	456	7,284	7,168	3.0	12.8	97.0	214.0
540	540	7,824	7,708	1.0	15.0	98.0	229.0
640	640	8,464	8,348	0.0	16.0	98.0	245.0
560	560	9,024	8,908	0.0	16.0	98.0	261.0
324	324	9,348	9,232	2.0	14.0	100.0	275.0
248	402	9,596	9,634	0.0	10.9	100.0	285.9
580	638	10,176	10,272	0.0	16.0	100.0	301.9
630	630	10,806	10,902	0.0	16.0	100.0	317.9
624	624	11,430	11,526	0.0	16.0	100.0	333.9
566	566	11,996	12,092	0.0	16.0	100.0	349.9
504	504	12,500	12,596	0.0	16.0	100.0	365.9
608	608	13,108	13,204	0.0	16.0	100.0	381.9
356	356	13,464	13,560	1.0	9.0	101.0	390.9
616	616	14,080	14,176	0.0	16.0	101.0	406.9
628	628	14,708	14,804	0.0	16.0	101.0	422.9
320	320	15,028	15,124	0.0	8.0	101.0	430.9
448	448	15,476	15,572	0.0	16.0	101.0	446.9
234	234	15,710	15,806	0.0	6.2	101.0	453.1
640	640	16,350	16,446	0.0	16.0	101.0	469.1

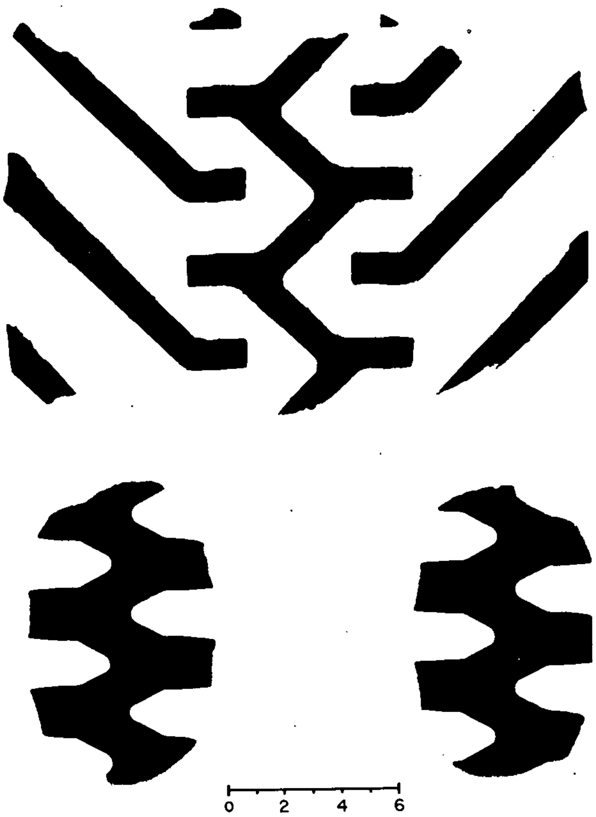


Figure 9. Contact prints for LPLS (top) and conventional (bottom) tires (actual contact area for the LPLS tire approximately 30-40 percent of gross contact area).

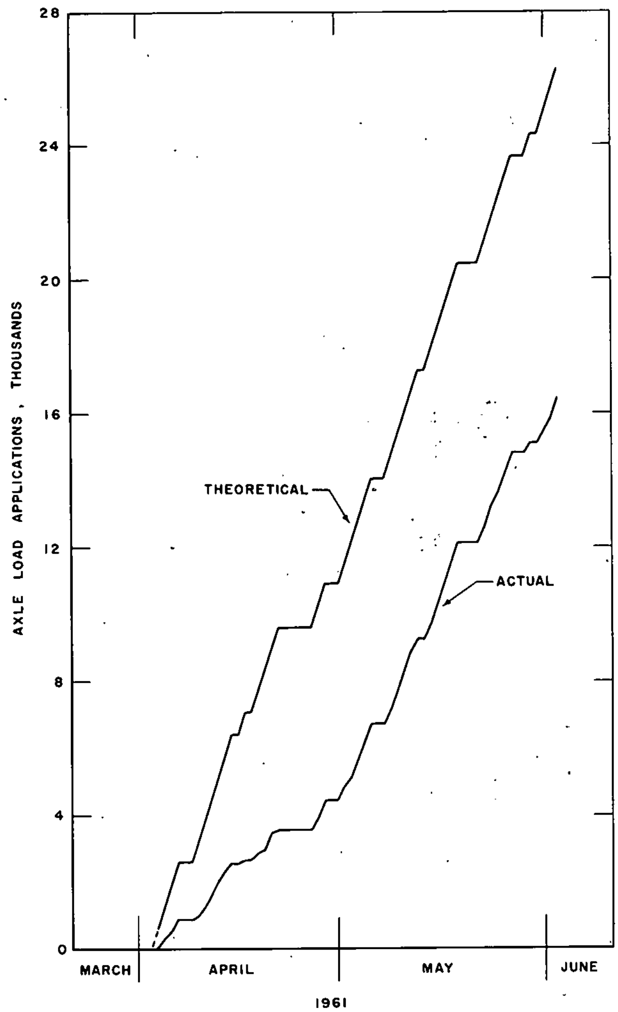


Figure 10. Cumulative axle load applications.

traffic as described in AASHO Road Test Report 5. These included slope variance (a measure of roughness), rut depths, extent of cracking and patching, pavement structure properties, deflections and strains. However, pavement distress occurred so rapidly in some cases that it was not possible to obtain all the information desired.

The performance of a pavement at the Road Test was considered to be represented by the trend of the pavement's serviceability with load applications. Serviceability, or ability to serve traffic, was represented on a numerical scale from 0 to 5 with adjective designations of very poor (0-1), poor (1-2), fair (2-3), good (3-4), and very good (4-5). Those unfamiliar with this concept may obtain a perspective from the following: good new pavements have serviceabilities of about 4.5, and most drivers will reduce their speed on a pavement if its serviceability is less than 2.0. The serviceability concept is described in detail in a paper in HRB Bulletin 250. An adaptation of this paper is also reproduced as an Appendix in AASHO Road Test Report 5. Present serviceability of each of the Road Test sections was determined every 2 weeks during the main traffic test. Sections were considered unsafe for traffic and were removed from testing and repaired when their serviceability dropped to 1.5. During the special studies, serviceabilities were determined more frequently.

Serviceability index values were obtained from formulas (one for flexible pavement and one for rigid pavement) that combined functions of longitudinal and transverse pavement profile and cracking and patching measured from the pavement surfaces. The index formulas were derived by means of an analysis in which subjective ratings of the serviceabilities of 74 flexible and 49 rigid pavements in actual service were correlated with data from measurements taken on the same pavements. An additional 15 rigid pavements were rated in order to confirm the rigid pavement formula.

All measurements and observations necessary for the determination of the serviceabilities of the test sections in this study are

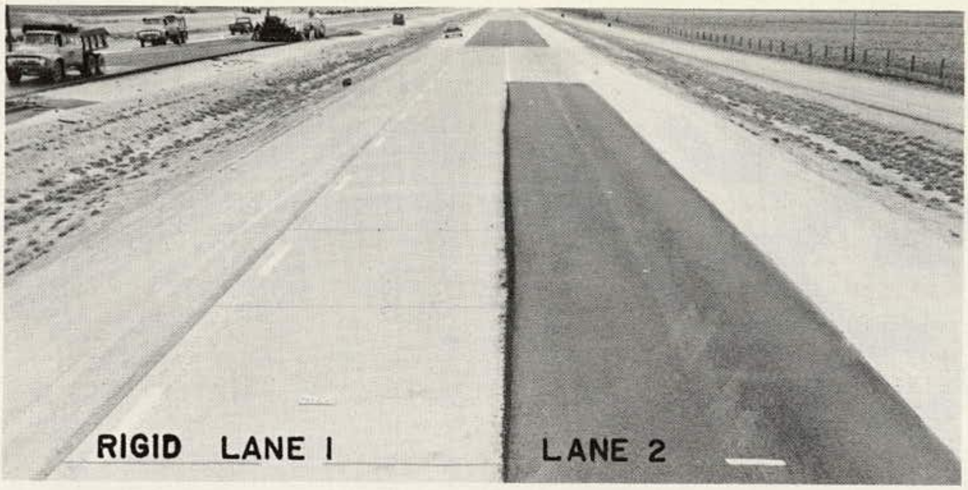


Figure 11. Rigid and flexible tangents, Loop 2, prior to performance study.

Figure 12. Traffic lane change guide lines on flexible tangent, Loop 2.

TABLE 5

SERVICEABILITY HISTORY

Section	Pavement Lane	Design	Tire Design	Original Serviceability	Serviceability Index After Applications Indicated: 338	854	962	1,636	2,874	3,494	4,082	5,046	6,284	8,514	10,456	14,230	16,500
(a) FLEXIBLE SECTIONS																	
711	1	2–6–4	Conv.	3.7	3.6	3.0	3.0	1.7									
712	2	2–6–4	LPLS	3.5	3.1	3.0	2.9	2.9	1.5	0.8							
737	1	2–6–4	LPLS	3.8	3.5	3.5	3.4	2.9	1.3	1.0							
738	2	2–6–4	Conv.	3.7	2.5	1.6	3.3	2.6	1.6	0.8							
745	1	3–3–4	LPLS	3.4	3.2	3.4	2.3	0.2									
746	2	3–3–4	Conv.	2.6	2.2	0.7											
749	1	3–6–0	LPLS	3.7	3 8	3.7	3.7	3.0	1.5								
750	2	3–6–0	Conv.	3.5	3.1	2.7	1.9	0.6									
757	1	2–6–0	LPLS	4.1	4.0												
758	2	2–6–0	Conv.	1.6													
761	1	0–6–4	LPLS	2.3	2.2												
762	2	0–6–4	Conv.	1.8													
763	1	3–6–4	LPLS	3.8	3.4	3.8	3.3	3.0	3.1	2.4	2.7	2 8	2.7	2.4	1.5		
764	2	3–6–4	Conv.	3.7	3.6	3.6	3.1	2.7	3.7	2.6	3.0	2.1	2.6	1.7	0.6		
775	1	2–6–0	LPLS	3.9	3.5	3.0	0.6										
776	2	2–6–0	Conv.	3.5	3.4	3.1	1.5										
(b) RIGID SECTIONS																	
787	1	3.5–6	LPLS	4.3	4.0	4.4	4 5	4.4	4.5	3.5	3.3	—					
788	2	3.5–6	Conv.	4.2	4.2	4.2	4.2	3.3	—								
793	1	3.5R–0	LPLS	4.4	4.3	4.4	4.4	4.3	4.5	4.4	4.4	4.3	4.4	4.2	3.6	—	
794	2	3.5R–0	Conv.	4.6	4.3	4.4	4.4	4.2	3.4	4.1	3.9	2.7	—				
811	1	3.5–3	LPLS	4.0	3.8	4.1	4.2	—									
812	2	3.5–3	Conv.	4.6	4.1	4.2	4.5	4.4	4.5	4.3	3.2	2.1	—				

reported in this section. Since many of the test sections in Loop 2 had failed prior to this study, a balanced factorial experiment design could not be used. Thus, a mathematical analysis of the performance data of the type that was reported for the main Road Test experiment could not be made. However, the performance equations developed in the main Road Test were applied to the conditions of this study. The results indicated that according to the equations the sections were expected to survive, in most cases, as many as ten times the applications that they did survive in the special studies.

However, the actual performance in the main Road Test of similar pavement sections in Loop 4 which were subjected to the 32-kip tandem axle loads agreed very well with their perform-

TABLE 6

ORIGINAL FLEXIBLE SECTIONS

Section	Lane	Section Design	Tire Design [1]	Serviceability [2]	Applications [3]
711	1	2–6–4	Conv.	3.7	1,850 [4]
712	2	2–6–4	LPLS	3.5	3,100
737	1	2–6–4	LPLS	3.8	2,680
738	2	2–6–4	Conv.	3.7	3,000
745	1	3–3–4	LPLS	3.4	1,350
746	2	3–3–4	Conv.	2.6	600
749	1	3–6–0	LPLS	3.7	2,900
750	2	3–6–0	Conv.	3.5	1,220
757	1	2–6–0	LPLS	4.1	750 [4]
758	2	2–6–0	Conv.	1.6	<350 [4]
761	1	0–6–4	LPLS	2.3	650 [4]
762	2	0–6–4	Conv.	1.8	500 [4]
763	1	3–6–4	LPLS	3.8	10,150
764	2	3–6–4	Conv.	3.7	8,900
775	1	2–6–0	LPLS	3.9	950
776	2	2–6–0	Conv.	3.5	980
717	1	1–3–4	LPLS	2.9	— [5]
755	1	1–6–0	LPLS	3.0	— [5]
759	1	2–3–0	LPLS	4.1	— [5]

[1] Conventional tires, 70 psi; LPLS tires, 35 psi.
[2] Prior to special study performance traffic.
[3] Applications to final seviceability of 1.5.
[4] Extrapolated.
[5] Special sections, serviceability below 1.5 before start of performance traffic.

TABLE 7

COMPARABLE ORIGINAL FLEXIBLE SECTIONS

Section	Lane	Section Design	Tire Design	Serviceability [1]	Applications [2]
711	1	2–6–4	Conv.	3.7	1,850 [3]
712	2	2–6–4	LPLS	3.5	3,100
737	1	2–6–4	LPLS	3.8	2,680
738	2	2–6–4	Conv.	3.7	3,000
749	1	3–6–0	LPLS	3.7	2,900
750	2	3–6–0	Conv.	3.5	1,220
763	1	3–6–4	LPLS	3.8	10,150
764	2	3–6–4	Conv.	3.7	8,250
775	1	2–6–0	LPLS	3.5	950
776	2	2–6–0	Conv.	3.5	980

[1] Prior to special study performance traffic.
[2] Applications to final serviceability of 1.5.
[3] Extrapolated.

TABLE 8

SERVICEABILITY LOSS OF 2.0 INDEX POINTS, FLEXIBLE SECTIONS

Section	Lane	Design	Tire Design	Serviceability Original	Serviceability Final	Applications [1]
711	1	2–6–4	Conv.	3.7	1.7	1,740
712	2	2–6–4	LPLS	3.5	1.5	3,100
737	1	2–6–4	LPLS	3.8	1.8	2,450
738	2	2–6–4	Conv.	3.7	1.7	2,650
745	1	3–3–4	LPLS	3.4	1.4	1,850
746	2	3–3–4	Conv.	2.6	0.6	800
749	1	3–6–0	LPLS	3.7	1.7	2,750
750	2	3–6–0	Conv.	3.5	1.5	1,220
757	1	2–6–0	LPLS	4.1	2.1	700
758	2	2–6–0	Conv.	1.6	—	— [2]
761	1	0–6–4	LPLS	2.3	0.3	850
762	2	0–6–4	Conv.	1.8	—	— [2]
763	1	3–6–4	LPLS	3.8	1.8	9,800
764	2	3–6–4	Conv.	3.7	1.7	8,400
775	1	2–6–0	LPLS	3.9	1.9	900
776	2	2–6–0	Conv.	3.5	1.5	980

[1] Applications to 2.0 loss of serviceability.
[2] Original serviceability less than 2.0.

TABLE 9

AREA OF CRACKING AND PATCHING, FLEXIBLE SECTIONS

Lane	Design	Tire Design	Total Cracking and Patching (sq ft/section) Initial	Final	Applications Sustained
1	2–6–4	Conv.	0	577	1,636
2	2–6–4	LPLS	20	77	1,636
1	2–6–4	LPLS	0	27	1,636
2	2–6–4	Conv.	5	115	1,636
1	3–3–4	LPLS	0	7	854
2	3–3–4	Conv.	35	97	854
1	3–6–0	LPLS	0	90	1,636
2	3–6–0	Conv.	0	310	1,636
1	2–6–0	LPLS	0	0	338
2	2–6–0	Conv.	463	556	338
1	0–6–4	LPLS	0	419	854
2	0–6–4	Conv.	207	620	854
1	3–6–4	LPLS	0	280	10,456
2	3–6–4	Conv.	0	1,048	10,456
1	2–6–0	LPLS	0	433	962
2	2–6–0	Conv.	20	489	962

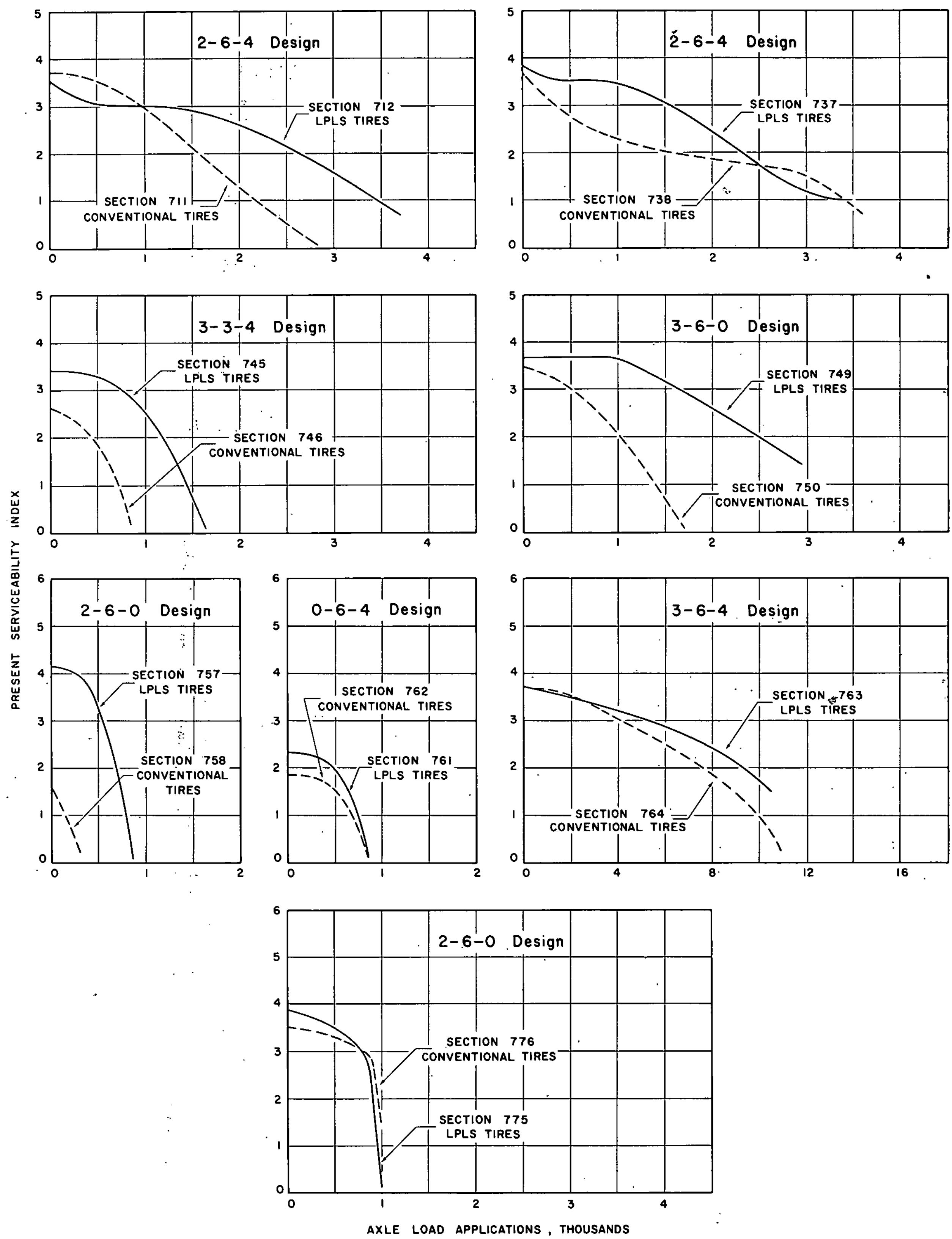


Figure 13. Serviceability trends, Loop 2, flexible sections.

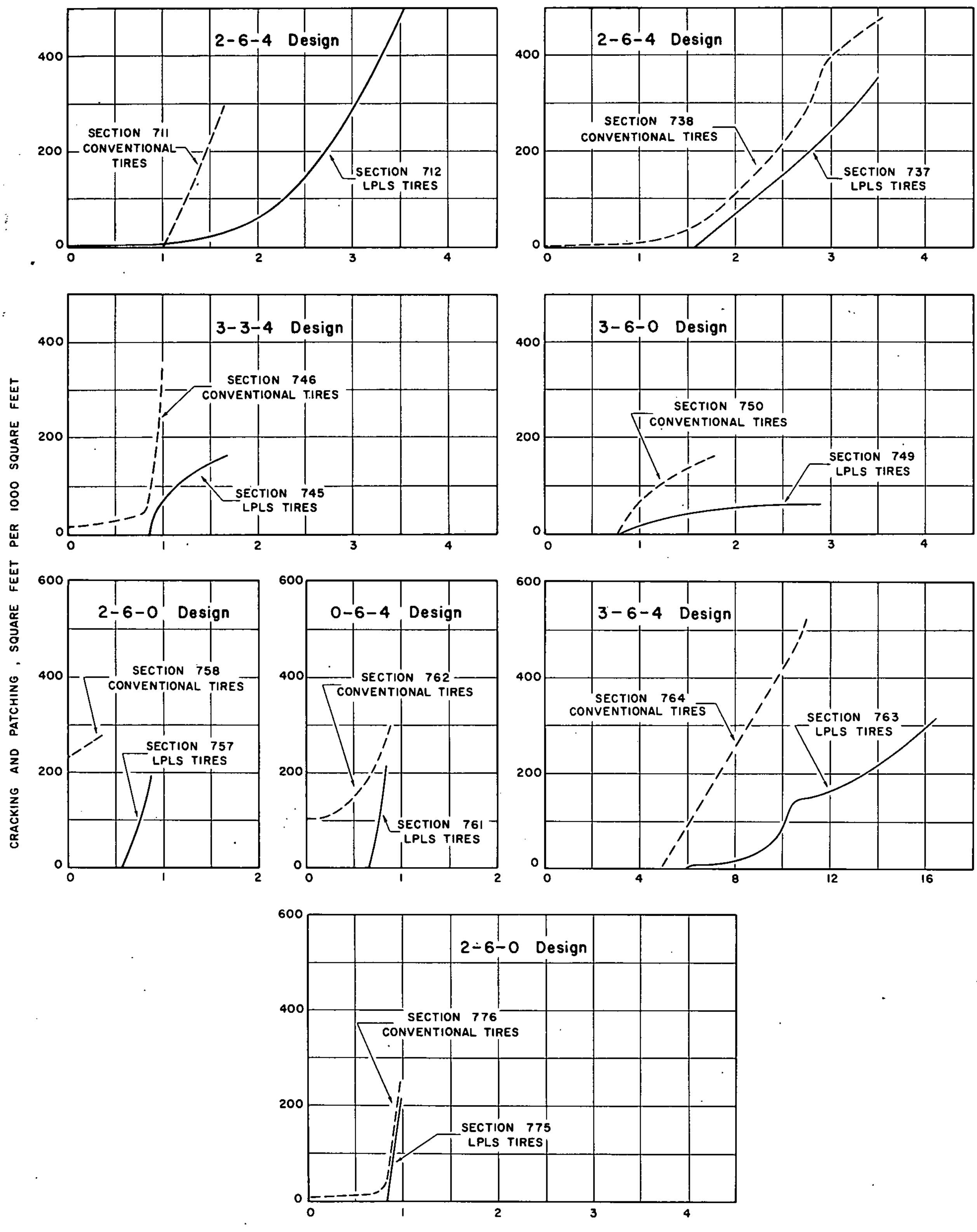


Figure 14. Cracking and patching trends, Loop 2, flexible sections.

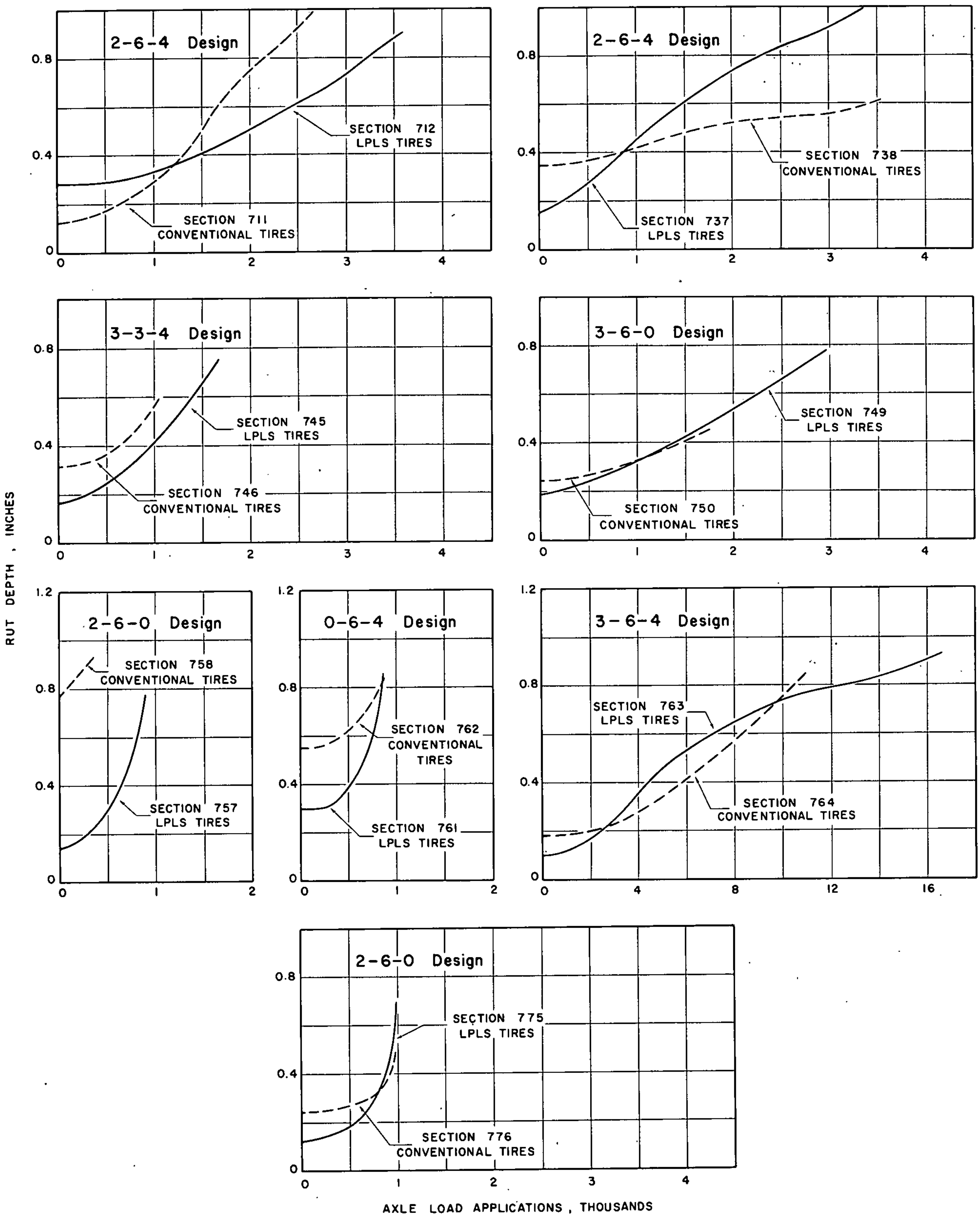


Figure 15. Rut depth trends, Loop 2, flexible sections.

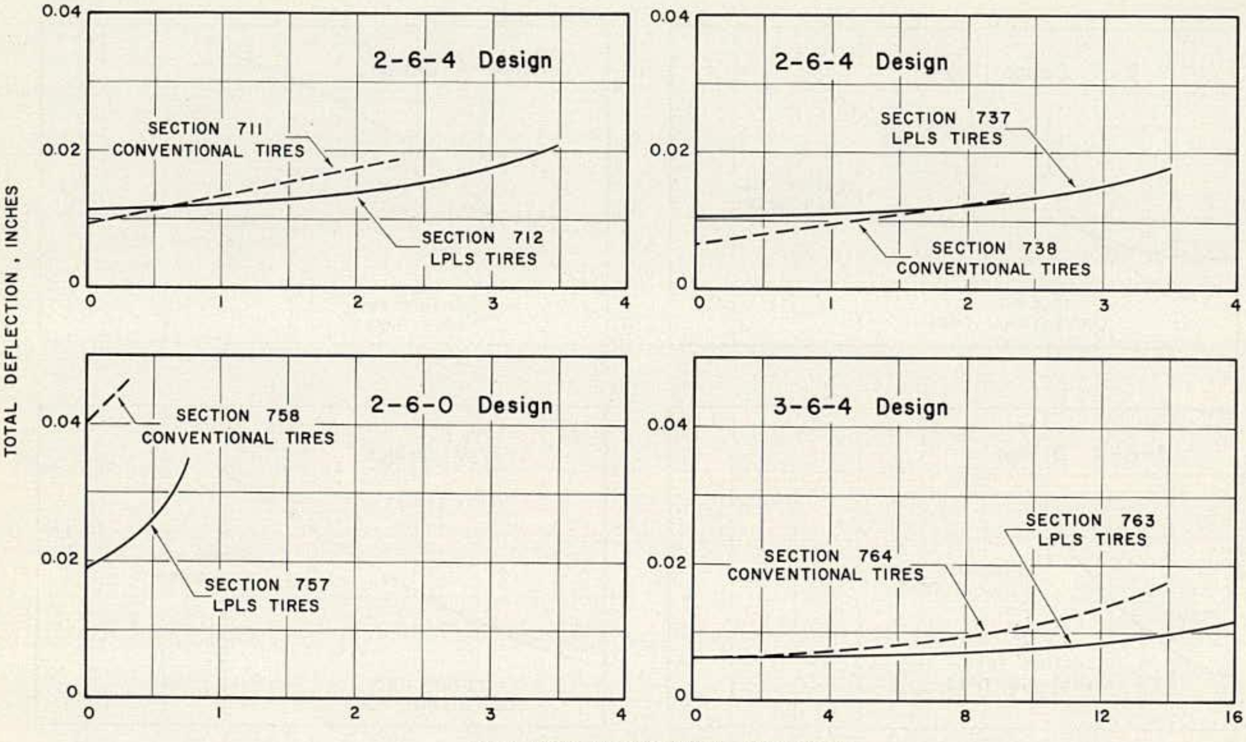


Figure 16. Deflection trends, Loop 2, flexible sections.

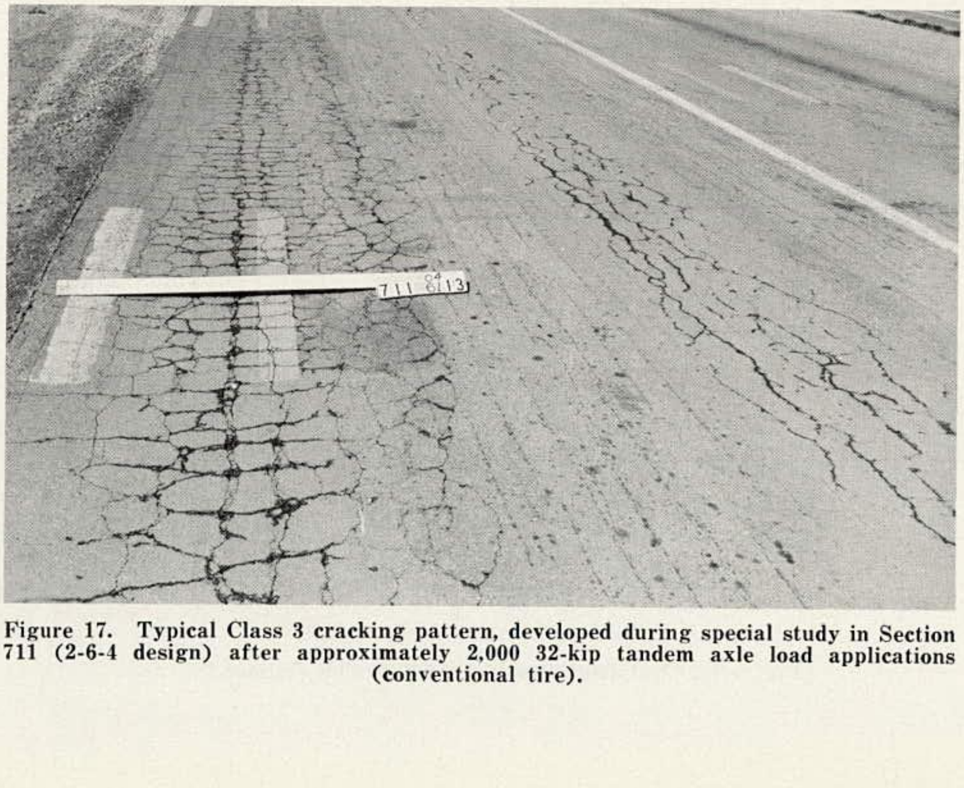


Figure 17. Typical Class 3 cracking pattern, developed during special study in Section 711 (2-6-4 design) after approximately 2,000 32-kip tandem axle load applications (conventional tire).

TABLE 10

Rut Depth Increase, Flexible Sections

Lane	Pavement Design	Tire Design	Original Depth of Rut (in.)	Rut Depth (in.) After Applications Indicated:										
				338	854	962	1,636	2,874	3,494	4,082	5,046	6,284	8,514	10,456
1	2–6–4	Conv.	0.14	0	0.10	0.13	0.48							
2	2–6–4	LPLS	0.30	0	0	0.10	0.14	0.40						
1	2–6–4	LPLS	0.15	0.10	0.23	0.37	0.49							
2	2–6–4	Conv.	0.35	0	0.05	0.09	0.17	0.20						
1	3–3–4	LPLS	0.15	0.09	0.17	0.24	0.26							
2	3–3–4	Conv.	0.32	0.01	0.15									
1	3–6–0	LPLS	0.18	0.04	0.14	0.14	0.26	0.57						
2	3–6–0	Conv.	0.29	0	0	0.06	0.08							
1	2–6–0	LPLS	0.12	0.07	0.13									
2	2–6–0	Conv.	0.77	0.17										
1	0–6–4	LPLS	0.30	0	0.55									
2	0–6–4	Conv.	0.73	0	0.12									
1	3–6–4	LPLS	0.10	0	0	0	0.07	0.12	0.23	0.34	0.34	0.42	0.52	0.67
2	3–6–4	Conv.	0.17	0	0.02	0.02	0.03	0.08	0.10	0.18	0.15	0.38	0.40	0.68
1	2–6–0	LPLS	0.07	0.05	0.28	0.61								
2	2–6–0	Conv.	0.24	0.01	0.11	0.24								

ance predicted from the equations. Therefore, it must be assumed that the conditions of embankment, etc., that obtained during this study were severe compared to those during the main Road Test. The data are displayed in the form of historical records for each of the sections. Table 5 gives the serviceability histories for the rigid and flexible sections that showed distress during the study.

2.4 FLEXIBLE PAVEMENT STUDY

Table 6 lists the flexible pavement sections studied, the designs of the sections and certain details of their condition at the beginning and end of the special tests. There are notations on the additional sections observed.

Factors in the index formula for determining the serviceability of the flexible pavement sections were the slope variance (a summary sta-

Figure 18. Typical well-developed ruts in outer wheelpath, lane 1.

TABLE 11

PERFORMANCE OF REPLICATE FLEXIBLE SECTIONS 711–712 AND 737–738; 2–6–4 DESIGN

(See Tables 7 to 10)

Lane	Tire Design	Original Serviceability	Applications: To Serviceability of 1.5	Applications: To Serviceability Loss of 2.0	Area of Cracking or Patching (sq ft): Original	Area of Cracking or Patching (sq ft): Increase[1]	Depth of Rut (in.): Original	Depth of Rut (in.): Increase[1]
1	Conv.	3.7	1,850	1,740	0	577	0.14	0.48
	LPLS	3.8	2,680	2,450	0	27	0.15	0.49
2	Conv.	3.7	3,000	2,650	5	115	0.35	0.17
	LPLS	3.5	3,100	3,100	20	77	0.30	0.14

[1] During special studies.

TABLE 12

ORIGINAL RIGID SECTIONS

Section	Lane	Design	Serviceability Prior to Traffic	Tire Design	Applications to Final Serviceability of 1.5	Serviceability at 16,500 Applications
777	1	5–3	3.6	LPLS	>16500	3.6
778	2	5–3	4.1	Conv.	>16500	3.7
779	1	3.5R–3	4.6	LPLS	>16500	4.5
780	2	3.5R–3	4.6	Conv.	>16500	2.8
783	1	3.5R–6	4.5	LPLS	>16500	4.5
784	2	3.5R–6	4.8	Conv.	>16500	4.0
785	1	2.5–6	4.5	LPLS	—[1]	
787	1	3.5–6	4.3	LPLS	4000[2]	
788	2	3.5–6	4.2	Conv.	1500[2]	
793	1	3.5R–0	4.4	LPLS	15000[2]	
794	2	3.5R–0	4.6	Conv.	6150[2]	
795	1	5R–6	4.2	LPLS	>16500	4.2
796	2	5R–6	4.3	Conv.	>16500	4.3
797	1	5–3	4.0	LPLS	>16500	3.8
798[3]	2	5–3	4.3	Conv.	>16500	4.0
799	1	2.5R–3	4.3	LPLS	—[1]	
801	1	5–0	4.1	LPLS	>16500	3.9
802	2	5–0	4.0	Conv.	>16500	3.9
803	1	5–6	4.0	LPLS	>16500	4.0
804	2	5–6	4.2	Conv.	>16500	3.9
807	1	5R–0	4.3	LPLS	>16500	4.2
808	2	5R–0	4.8	Conv.	>16500	4.3
809	1	5R–3	4.5	LPLS	>16500	4.5
810	2	5R–3	4.3	Conv.	>16500	4.3
811	1	3.5–3	4.0	LPLS	3900[2]	
812	2	3.5–3	4.6	Conv.	6850[2]	
815	1	3.5R–3	3.8	Conv.	>16500	4.1
816	2	3.5R–3	3.8	LPLS	>16500	3.9

[1] Special section, serviceability below 1.5 before start of performance traffic.

[2] Extrapolated.

[3] One-half of section in test.

tistic of longitudinal profile), depth of rut, and area of cracking and patching. Figures 13 through 16 show these measurements as well as the deflections measured in the flexible sections. The slope variance is expressed in slope units as determined by the Road Test profilometer; the depth of rut is in inches measured from a 4-ft reference; and the cracking and patching are expressed in square feet per 1,000 sq ft of pavement area. (See Road Test Report 5 for a detailed discussion of the serviceability index formula for flexible pavements.)

Making use of the data in Table 6 and Figures 13 through 16 a comparison could be made of the performance of the lane 1 sections, which were subjected to 32-kip tandem axle loads by vehicles equipped with LPLS tires, and lane 2 sections subjected to the same loading by vehicles equipped with conventional tires.

These data are summarized in Table 7 from which a direct comparison was made of the possible effect of the two different tire designs and inflation pressures on the serviceabilities of certain pairs of sections. The sections selected were those which had comparable serviceability indexes (within a few tenths of a point) at the beginning of the special study.

In three of the five instances, the sections subjected to the LPLS tires withstood more applications before the serviceability dropped to 1.5 than did their companion sections which were subjected to the conventional tires. However, in the remaining two instances the differences in applications to a serviceability level of 1.5 were so small that they could not be considered significant.

The sections not listed in Table 7 had different before-traffic serviceabilities. To include such sections in the study, a comparison was made based on the number of applications of load necessary to reduce the beginning serviceability by 2.0 index points, regardless of the level of beginning serviceability. These values were determined for each section by extrapolation from the historical plots and are listed in Table 8.

On this basis of comparison, the sections subjected to the conventional tires showed a greater rate of loss of serviceability for four of the eight pairs of sections in the study. Two pairs of the sections in the test could not be compared on this basis since in each pair one of the sections had an initial serviceability lower than 2.0. No analysis of the loss rate of serviceability was made from which it could be shown whether or not a section with a lower serviceability at the beginning of the traffic would show distress more rapidly than would a section of equal design with a higher beginning serviceability.

The rate of development of the area of cracking and patching was investigated in a further attempt to relate the effect of the tire design to the performance of the section. Figure 17 shows a typical cracking pattern for a flexible section.

Table 9 gives the total area of cracking and patching for each pair of test sections at a number of applications common to both sections.

In the eight pairs of sections, six sections subjected to the conventional tires showed a greater increase in total area of cracking and patching than did their companion sections subjected to LPLS tires. The two remaining pairs of sections showed a nearly equal increase in cracking and patching when compared at a common number of applications. It must be pointed out, however, that none of the lane 1 sections (LPLS tires, except section 711) had shown any previous distress in the form of cracking or patching under the 2-kip loads applied during the regular traffic test, whereas there had been considerable cracking under the 6-kip loads of lane 2. In other words, the before-traffic cracking and patching condition was different for each pair of sections.

Since rut depth appears in the serviceability formula, a comparison of the increase in depth of rut for each pair of sections for any given number of applications might be expected to indicate the relative effect of tire design and pressure.

Table 10 shows the original depth of rut and the increase in depth for each of the sections at various levels of axle applications. Figure 18 shows a typical rut developed under the 32-kip tandem axle loads.

In all pairs of sections the original depth of rut was greater in the lane 2 sections which had carried the 6-kip axle load during the regular test traffic. During the special study the lane 2 sections (except section 712) were subjected to vehicles equipped with the conventional tires. However, in 4 of the 8 pairs of sections, the increase in rut depth was greater in the lane 1 sections subjected to the vehicles equipped with the LPLS tire. These sections (lane 1) had lower initial rut depths in all instances. Road Test Report 5 (Section 2.2.3) indicates the possibility that a certain depth of rut can develop after which the rate of increase declines.

Of the remaining four pairs of sections, two showed a greater increase in rut depth for the vehicles equipped with the conventional tires,

Figure 19. Distress caused by tire pressure-tire design study vehicle in Section 755 (1-6-0 design), less than 50 axle load application.

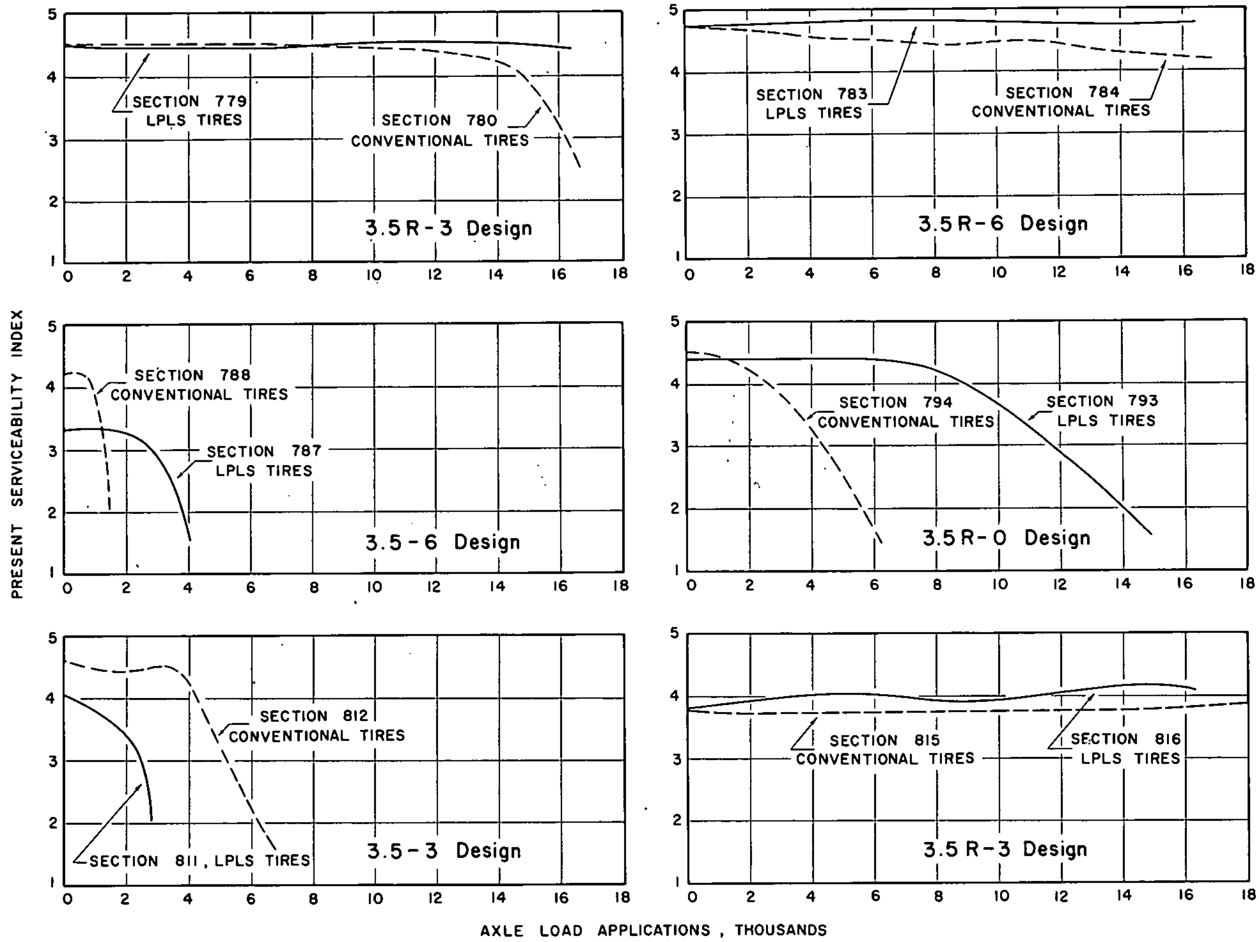


Figure 20. Serviceability trends, Loop 2, rigid sections.

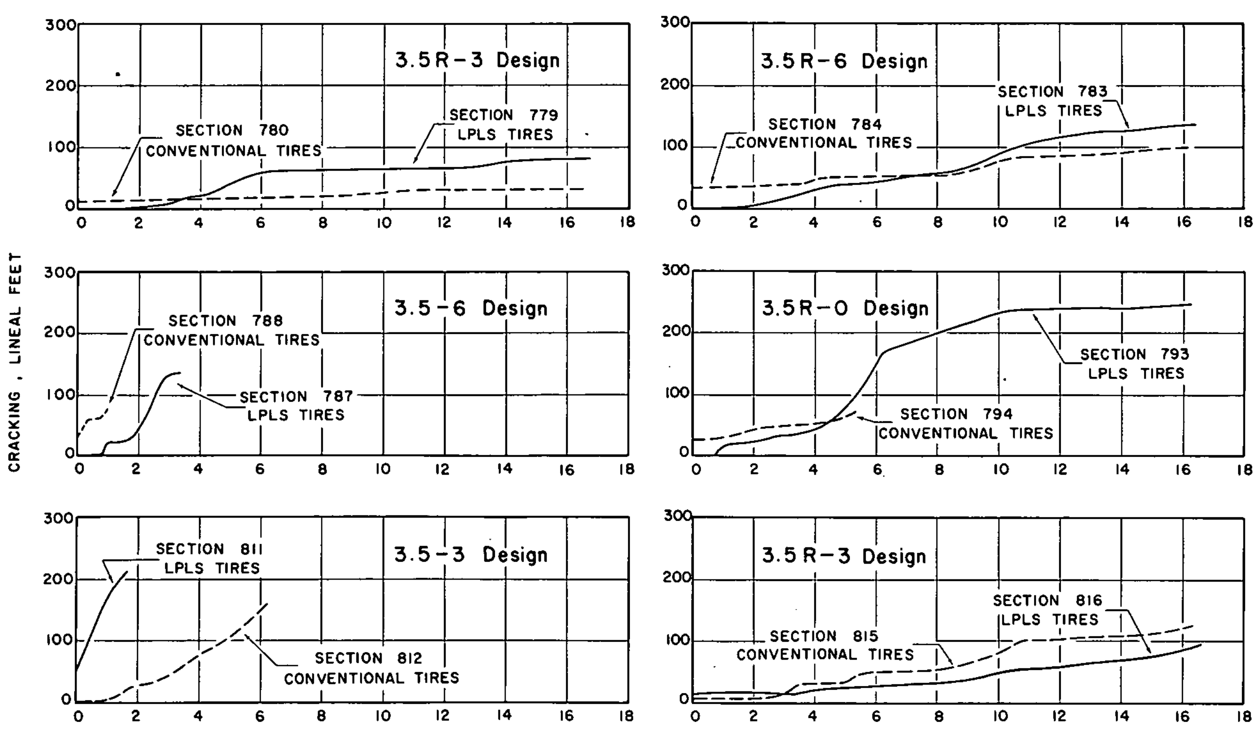


Figure 21. Cracking trends, Loop 2, rigid sections.

TABLE 13

CREEP SPEED DEFLECTION, RIGID SECTIONS

Lane	Tire Design	Design	Deflection (in. × 10^{-3}) for Applications and Pavement Temperature Indicated:																	
			0 43F		2,288 56F		3,444 67F		4,536 39F		6,234 44F		9,024 70F		11,996 60F		14,080 43F		16,350 78F	
			Corner	Edge	Corner	Edge	Corner	Edge	Corner	Edge	Corner	Edge	Corner	Edge	Corner	Edge	Corner	Edge	Corner	Edge
1	LPLS	3.5–3	63	58																
2	Conv.	3.5–3	41	29	103	45	63	40	74	44										
1	LPLS	3.5–6	35	32	32	25														
2	Conv.	3.5–6	20	24																
1	LPLS	5.0–0	40	36	46	34	40	32	43	32	54	29	44	33	45	31	32	34	50	33
2	Conv.	5.0–0	48	37	56	42	57	45	52	32	56	33	67	37	76	40	68	39	119	45
1	LPLS	5.0–3	37	32	41	31	35	27	34	28	48	36	38	32	33	24	34	22	37	28
2	Conv.	5.0–3	30	26	45	35	31	26	41	28	64	32	35	33	36	28	35	27	45	35
1	LPLS	5.0–3	37	33	43	31	32	25	28	24	39	28	28	25	30	27	34	26	37	27
2	Conv.	5.0–3	36	27	51	33	29	29	46	28	92	46	57	38	35	25	37	26	52	35
1	LPLS	5.0–6	36	30	37	28	29	25	26	26	30	23	32	29	29	25	31	25	29	24
2	Conv.	5.0–6	31	26	57	38	37	29	48	30	93	37	49	29	44	31	54	28	50	30
1	LPLS	3.5R–0	54	35	39	30	43	32	51	39										
1	Conv.	3.5R–3	42	41	55	31														
2	LPLS	3.5R–3	50	34	90	44	53	38	57	42	114	44	82	35						
1	LPLS	3.5R–3	36	35	50	32														
2	Conv.	3.5R–3	37	30																
1	LPLS	3.5R–6	37	34	38	32	31	25	38	27	47	23								
1	LPLS	5.0R–0	36	32	38	30	41	35	38	34	41	28	39	30	44	29	41	27	48	32
2	Conv.	5.0R–0	39	33	47	34	37	27	50	35	67	37	38	31	43	31	38	32	55	33
1	LPLS	5.0R–3	42	32	38	29	28	25	37	32	40	27	35	30	33	25	37	27	34	27
2	Conv.	5.0R–3	36	25	49	34	31	27	40	26	74	29	33	27	38	30	48	24	41	27
1	LPLS	5.0R–6	33	31	34	29	29	27	35	27	49	29	28	22	28	23	35	25	38	25
2	Conv.	5.0R–6	40	28	40	43	34	24	51	34	88	33	51	33	44	29	52	22	51	30

TABLE 14

INCREASE IN LINEAL FEET OF CRACKING, RIGID SECTIONS

Lane	Pavement Design	Tire Design	Cracking[1] (lin ft) for Applications Indicated:													
			0	338	854	962	1,636	2,874	3,494	4,082	5,046	6,284	8,514	10,456	14,230	16,500
1	3.5R–0	LPLS	0	0	0	17	21	33	35	45	81	171	210	237	240	246
2	3.5R–0	Conv.	28	2	2	2	12	21	23	25	41					
1	3.5–3	LPLS	58	37	88	107	156									
2	3.5–3	Conv.	2	0	0	3	25	36	59	78	108	163				
1	3.5R–3	LPLS	0	0	0	0	0	7	20	21	46	62	63	64	78	79
2	3.5R–3	Conv.	13	0	0	0	0	1	2	2	6	6	9	17	18	18
1	3 5R–3	Conv.	7	0	0	0	0	4	23	24	30	41	49	88	96	119
2	3.5R–3	LPLS	13	0	0	0	0	2	2	10	21	21	21	40	57	80
1	3.5R–6	LPLS	3	0	0	0	0	12	18	29	33	41	56	94	121	131
2	3.5R–6	Conv.	34	0	0	0	0	5	5	16	16	18	20	52	59	66
1	3.5–6	LPLS	0	1	1	21	24	131	138							
2	3.5–6	Conv.	32	28	31	46										
1	5–0	LPLS	0													
2	5–0	Conv.	0													
1	5R–0	LPLS	0													
2	5R–0	Conv.	0													
1	5–3	LPLS	0													
2	5–3	Conv.	0													
1	5–3	LPLS	0													
2	5–3	Conv.	0													
1	5R–3	LPLS	0													
2	5R–3	Conv.	0													
1	5–6	LPLS	0													
2	5–6	Conv.	0													
1	5–6	LPLS	0													
2	5–6	Conv.	0													

[1] For all 5-in. sections the total increase in lineal feet of cracking, all classes, was less than 100 ft.

TABLE 15

INCREASE IN PUMPING SCORE, RIGID SECTIONS

Lane	Pavement Design	Tire Design	Initial Pumping Score	Pumping Score for Applications Indicated:		
				1,636	2,674	7,334
1	5 –3	LPLS	0	0	0	0
2	5 –3	Conv.	569	23	84	184
1	3.5R–3	LPLS	0	2	2	3
2	3.5R–3	Conv.	1,204	0	13	96
1	3.5R–6	LPLS	0	2	2	25
2	3.5R–6	Conv.	748	29	252	512
1	2.5 –6	LPLS	0	0	0	0
1	3.5 –6	LPLS	0	6	6	6
2	3.5 –6	Conv.	1,322	77	77	77
1	3.5R–0	LPLS	0	0	0	0
2	3.5R–0	Conv.	821	0	19	19
1	5R–6	LPLS	0	0	0	10
2	5R–6	Conv.	194	63	211	383
1	5 –3	LPLS	0	0	0	28
2	5 –3	Conv.	634	7	16	50
1	2.5R–3	LPLS	0	0	0	0
1	5 –0	LPLS	0	0	0	0
2	5 –0	Conv.	421	5	11	29
1	5 –6	LPLS	0	0	0	39
2	5 –6	Conv.	222	104	405	583
1	5R–0	LPLS	0	0	0	0
2	5R–0	Conv.	364	0	0	11
1	5R–3	LPLS	0	0	0	0
2	5R–3	Conv.	459	1	16	53
1	3.5 –3	LPLS	0	7	7	7
2	3.5 –3	Conv.	712	10	240	240
1	3.5R–3	Conv.	0	0	0	5
2	3.5R–3	LPLS	1,015	0	13	68

TABLE 16

PERFORMANCE OF REPLICATE RIGID SECTIONS 779–780 AND 815–816; 3.5R–3 DESIGN

(See Tables 12, 14 and 15)

Lane	Tire Design	Pumping Score		Cracking (lin ft)	
		Original	Increase[1]	Original	Increase[1]
1	Conv.	0	5	7	0
	LPLS	0	3	119	79
2	Conv.	1,204	96	13	13
	LPLS	1,015	68	18	80

[1] During special studies.

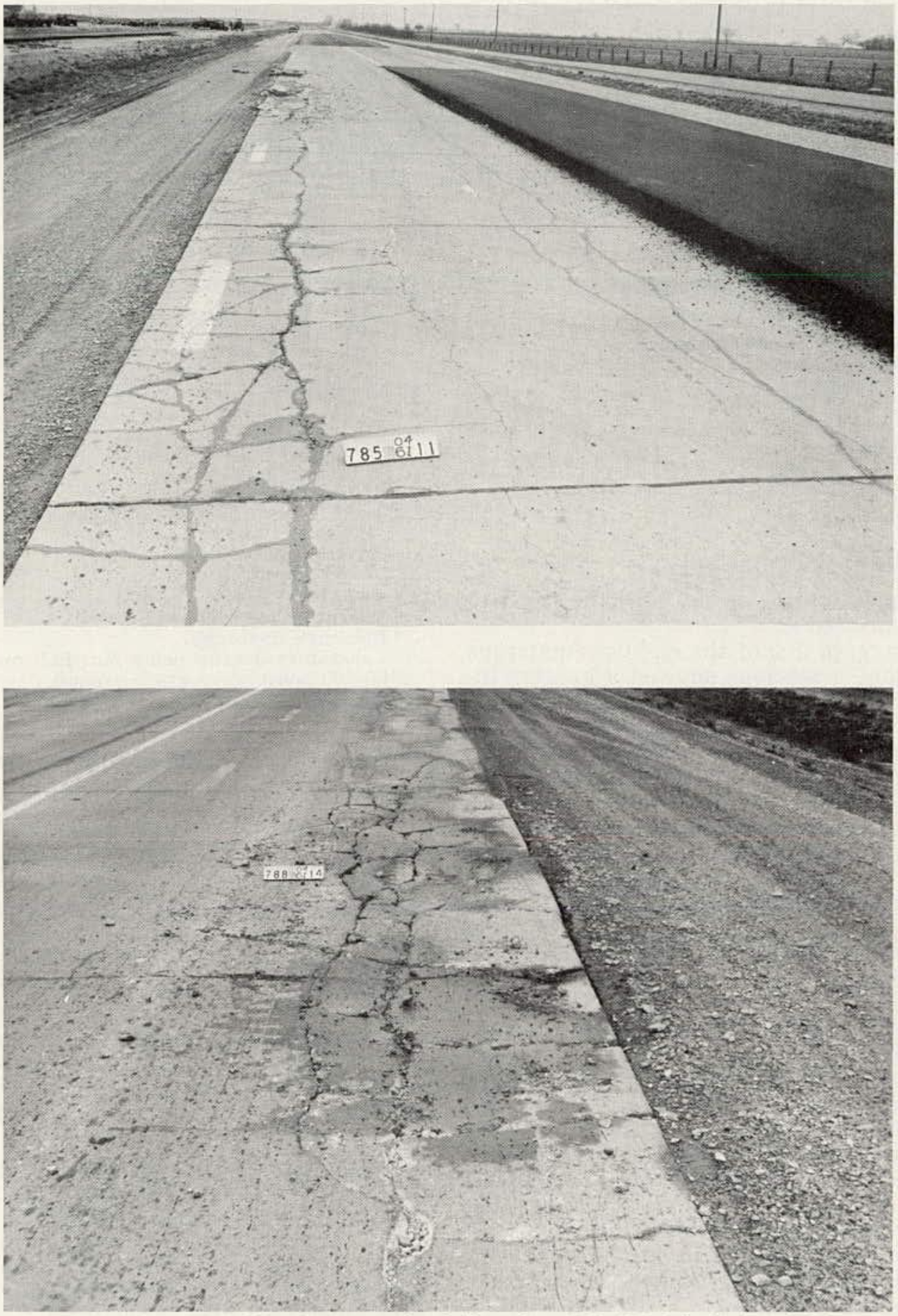


Figure 22. Typical crack pattern in rigid sections 785 (2.5-6 design) and 788 (3.5-6 design), lanes 1 and 2, respectively.

TABLE 17

CHARACTERISTICS OF EMBANKMENT SOIL

A. Classification (AASHO M-145)	A-6
B. Average values, borrow pit samples:	
Maximum dry density, AASHO T-99 (pcf)	116
Optimum moisture content (%)	15
Liquid limit (%)	29
Plasticity index	13
Grain size (% finer than):	
No. 200	81
0.02 mm	63
0.005 mm	42
Specific gravity	2.71
C. Average of construction tests:	
Density (% compaction)	97.7
Moisture content (%)	16
D. Tests on samples from constructed embankment:	
Flexible pavement:	
Laboratory CBR, soaked	2–4
Field in-place CBR[1]	2–4
Rigid pavement:	
Modulus of subgrade reaction,[1] k	45

[1] Spring condition.

TABLE 18

CHARACTERISTICS OF MATERIALS, FLEXIBLE PAVEMENTS

Item	Subbase[1]	Crushed Stone[2] Base	Asphaltic Concrete: Surface Mix	Asphaltic Concrete: Binder Mix
Aggregate gradation (% passing):				
1½ in.		100		
1 in.	100	90		100
¾ in.	96	80	100	
½ in.	90	68	92	75
No. 4	71	50	65	36
No. 40	25	21	22	13
No. 200	7	11	5	4
Plasticity index, minus No. 40 material	N.P.	N.P.		
Max. dry density (pcf)	138	139	151[3]	154[3]
Field density (% compaction)	102	102	97	97
Asphalt[4] content (% total mix)			5.4	4.5
Laboratory tests:				
Marshall stability			2,000	1,800
Marshall flow			11	11
Total voids			3.6	4.8

[1] Uncrushed natural sand-gravel.
[2] Dolomitic limestone.
[3] Laboratory density using Marshall procedure.
[4] 85–100 penetration grade asphalt.

and two showed little or no effect of the tire design and pressure.

In summary, in five of the eight comparisons made, the lane 1 sections showed a greater increase in rut depth than the lane 2 sections regardless of the tire design or pressure.

The comparisons for the replicate section on the basis of serviceability loss, increase in area of cracking and patching and increase in depth of rut are shown in Table 11. Although the differences are slight in most of the comparisons, those sections subjected to the LPLS tires, either lane 1 or lane 2, show a lower rate of distress, except for rut depth increase, than do their companion sections subjected to the conventional tire.

Three flexible pavement sections (717, 755 and 759) of the lightest design in Loop 2 were selected for special observation at the request of the Department of Defense. These sections were highly distressed under tire pressure-tire design traffic (Chapter 3) which was operated prior to the start of the performance study. Thus, little data were available on the performance of these sections. Figure 19 shows the early distress in section 759 which had an original design of 2-3-0 (2 in. of asphaltic concrete surfacing, 3 in. of crushed stone base, and no subbase).

2.5 RIGID PAVEMENT STUDY

Studies similar to those for flexible pavements were made for the rigid pavement sections included in the special performance tests. Table 12 gives the rigid sections studied, the designs of the sections, and certain details of their condition at the beginning and end of the special tests. There are notations on the additional sections observed.

In determining the serviceability of the rigid sections, the slope variance and extent of class 3 and 4 cracking* were factors in the index formula. Figures 20 and 21 show these measurements for all sections except the two lightest design sections (785 and 799) and sections with 5 in. of surfacing. Sections 785 and 799 were seriously distressed before the start of performance traffic. In the 5-in. surface sections, a total of less than 100 lin ft of cracking developed, and the loss of serviceability was negligible for the 16,500 axle loads applied.

Edge and corner deflections are not shown on the historical plots for the rigid sections because wide temperature effects were encountered and deflection is greatly influenced by slab temperature differential (see Road Test Report 5). However, Table 13 gives the average deflections for the sections.

Comparison of the effect of tire design on the rigid pavement sections, on the basis of

* Class 3 cracks are defined as any crack spalled at the surface to a width of 1/4 in. or more for at least one-half its length. Class 4 cracks are defined as any crack which has been sealed. (See Report 5, Chapter 3.)

TABLE 19

SUMMARY TRENCHING PROGRAM, FLEXIBLE SECTIONS

Section	Lane	Time[1]	Outer Wheelpath: Moist. Cont. (%)		Outer Wheelpath: Dry Density (pcf)	Outer Wheelpath: CBR (0.1 in.)	Outer Wheelpath: Plate Load Test[2] (lb/cu in.)		Between Wheelpaths: Moisture Content (%)				Between Wheelpaths: Dry Density (pcf)			Between Wheelpaths: CBR (0.1 in.)
							k_R	k_G								
(a) Embankment																
			0–1 In.	0–4 In.	0–4 In.				0–1 In.	0–4 In.	16 In.	30 In.	0–4 In.	16 In.	30 In.	
719	1	1	13.8	15.9	112.6	2.5	86	42	14.9	16.7	—	14.6	107.8	—	114.1	1.8
		2	13.0	15.0	112.3	2.7	94	64	14.4	16.2	19.0	14.6	110.3	104.1	112.7	2.3
741	1	1	15.3	16.6	109.8	1.8	86	39	16.1	16.8	17.0	15.1	107.9	107.4	114.4	1.3
		2	14.5	16.4	108.5	2.9	91	50	14.4	16.3	16.0	15.1	107.2	111.0	111.0	2.6
773	1	1	16.0	17.1	107.6	1.6	86	43	15.8	16.4	14.8	14.8	108.3	106.4	114.0	1.5
		2	16.1	17.0	107.8	2.8	89	46	17.1	16.9	16.5	15.8	108.0	—	111.7	2.3
Mean	1	1	15.0	16.5	110.0	2.0	86	41	15.6	16.6	15.9	14.8	108.0	106.9	114.2	1.5
		2	14.5	16.1	109.5	2.8	91	53	15.3	16.5	17.2	15.2	108.5	107.5	111.8	2.4
720	2	1	16.6	17.1	108.5	2.0	91	37	16.1	16.7	15.1	14.8	109.0	109.6	111.2	2.4
		2	15.0	16.0	111.0	2.8	101	69	14.7	15.1	16.5	15.5	112.2	107.6	113.5	2.6
742	2	1	15.1	17.5	109.3	1.1	94	37	15.2	16.9	16.5	15.1	110.0	109.4	113.7	1.8
		2	16.0	16.5	108.7	2.6	91	52	14.4	16.4	17.0	14.6	108.5	105.8	110.6	2.8
774	2	1	16.3	18.4	107.0	1.7	100	56	16.3	17.4	16.0	13.2	108.5	110.3	—	1.6
		2	15.7	17.4	108.2	2.8	98	58	15.9	17.0	14.6	13.6	108.2	113.2	117.5	1.8
Mean	2	1	16.0	17.7	108.2	1.6	95	43	15.9	17.0	15.9	14.4	109.2	109.8	112.4	1.9
		2	15.6	16.6	109.3	2.7	97	60	15.0	16.2	16.0	14.6	109.6	108.8	113.9	2.4
Mean	1 & 2	1	15.5	17.1	109.2	1.8	90	42	15.7	16.8	15.9	14.6	108.6	108.6	113.5	1.7
		2	15.1	16.4	109.4	2.8	94	56	15.2	16.3	16.6	14.9	109.1	108.3	113.1	2.4
(b) Base Course																
719	1	1	4.5		142.1	58			4.3				139.3			72
		2	4.0		142.3	88			4.2				136.6			52
741	1	1	4.7		132.6	27			4.6				134.0			20
		2	4.1		142.3	52			4.1				137.0			46
773	1	1	4.7		136.6	12			4.8				141.8			7
		2	4.3		140.6	12			4.3				139.2			19
Mean	1	1	4.6		137.1	32			4.6				138.4			33
		2	4.1		141.7	50			4.2				137.6			39
720	2	1	4.4		—	52			4.7				—			73
		2	3.8		136.3	108			3.8				137.6			103
742	2	1	4.7		137.2	26			4.7				137.6			27
		2	3.7		144.4	56			3.8				139.8			48
774	2	1	4.6		138.4	12			5.0				136.2			10
		2	4.0		135.1	17			4.0				135.1			11
Mean	2	1	4.6		137.8	30			4.8				136.9			36
		2	3.8		138.6	60			3.9				137.5			54
Mean	1 & 2	1	4.6		137.4	31			4.7				137.8			35
		2	4.0		140.2	55			4.0				137.5			46
(c) Subbase																
719	1	1	6.2		139.4	10.8			6.3				139.2			6.8
		2	5.3		140.2	10.3			6.2				138.0			7.5
741	1	1	6.0		137.8	6.4			6.0				138.4			12.0
		2	5.1		138.4	25.5			5.8				137.2			13.1
773	1	1														
		2														
Mean	1	1	6.1		136.4	8.6			6.1				138.8			9.4
		2	5.2		139.3	17.9			6.0				137.6			10.3
720	2	1	5.3		142.3	9.3			5.4				136.8			6.0
		2	5.0		136.2	16.7			4.9				137.7			9.8
742	2	1	5.5		142.2	13.1			5.9				143.4			11.5
		2	5.0		136.2	14.5			5.0				136.0			20.8
774	2	1														
		2														
Mean	2	1	5.4		142.2	11.2			5.6				140.1			8.8
		2	5.0		136.2	15.6			5.0				136.8			15.3
Mean	1 & 2	1	5.8		139.3	9.9			5.9				139.4			9.1
		2	5.1		137.8	16.8			5.5				137.2			12.8

[1] Time 1 = pre-performance traffic; Time 2 = post-performance traffic.
[2] 30-in. plate.

TABLE 20

SUMMARY OF TRENCHING PROGRAM, RIGID SECTIONS

Section	Lane	Time[1]	Inner Wheelpath						Outer Wheelpath				Between Wheelpaths						
			Moist. Cont. (%)		Dry Density (pcf)	CBR (0.1 in.)	Plate Load Test[2] (lb/cu in.)		Moist. Cont. (%)		Dry Density (pcf)	CBR (0.1 in.)	Moisture Content (%)				Dry Density (pcf)		
							k_E	k_G											
(a) Embankment																			
			0–1 In.	0–4 In.					0–1 In.	0–4 In.			0–1 In.	0–4 In.	16 In.	30 In.	0–4 In.	16 In.	30 In.
789	1	1	17.8	18.5	103.6	1.0	56	22	17.4	19.0	104.6	0.8	17.0	19.0	15.1	15.5	105.8	110.6	108.6
		2	16.8	18.8	105.0	1.5	62	32	16.6	17.6	106.5	1.6	16.2	17.7	16.0	16.0	106.4	111.6	111.9
799	1	1	17.8	18.3	104.0	0.8	49	17	17.8	18.7	105.7	—	16.6	18.9	17.0	15.1	106.2	108.1	109.6
		2	16.6	18.0	106.5	1.1	64	27	17.0	17.8	106.2	2.0	16.2	16.9	14.3	15.3	108.0	114.7	111.8
815	1	1	17.2	17.4	107.1	0.7	51	20	17.3	17.6	107.2	—	18.0	18.6	16.3	15.1	105.2	105.9	112.0
		2	16.2	17.3	108.4	1.4	68	34	16.5	18.0	106.6	0.9	16.9	18.1	15.8	15.3	105.8	109.1	111.9
Mean	1	1	17.6	18.1	104.9	0.8	52	20	17.5	18.4	105.8	—	17.2	18.8	16.1	15.2	105.7	108.2	110.0
		2	16.5	18.0	106.6	1.3	65	31	16.7	17.8	106.4	1.5	16.4	17.6	15.4	15.5	106.7	111.8	111.8
790	2	1	16.6	18.1	105.2	1.1	64	27	16.4	18.3	106.8	1.1	17.5	18.2	15.5	14.6	104.0	112.9	112.7
		2	16.4	17.6	107.2	1.4	71	37	16.0	18.6	107.7	2.0	15.0	18.1	14.3	15.8	107.9	114.4	—
800	2	1	17.4	17.2	109.3	0.7	64	21	18.3	18.8	108.3	—	18.0	18.7	18.0	17.0	103.6	109.0	104.7
		2	17.5	17.9	106.4	1.5	83	39	15.7	18.5	106.3	1.5	17.1	17.7	15.3	15.1	103.0	111.8	111.3
816	2	1	16.1	17.2	107.0	1.0	54	18	16.4	18.1	107.3	—	17.5	17.2	17.0	18.8	108.4	109.6	107.3
		2	15.9	18.6	105.8	1.1	65	27	16.4	18.2	105.5	1.3	17.5	18.0	15.3	15.1	105.5	107.5	110.7
Mean	2	1	16.7	17.5	107.2	0.9	61	22	17.0	18.4	107.4	—	17.7	18.0	16.8	16.8	105.3	110.5	108.2
		2	16.6	18.0	106.4	1.3	73	34	16.0	18.4	106.5	1.6	16.5	17.9	15.0	15.3	105.5	111.2	111.0
Mean	1 & 2	1	17.2	17.8	106.0	0.9	56	21	17.3	18.4	106.6	—	17.4	18.4	16.5	16.0	105.5	109.4	109.2
		2	16.6	18.0	106.6	1.3	69	33	16.4	18.1	106.5	1.6	16.5	17.8	15.2	15.4	106.1	111.5	111.5
(b) Subbase																			
789	1	1	8.0		136.1		70	27	7.5		135.2								
		2	7.3		135.1		82	38	7.3		134.7								
799	1	1	8.9		131.3		54	18	8.9		124.3								
		2	7.5		132.2		69	36	7.0		132.3								
815	1	1	9.2		133.0		58	19	9.3		134.0								
		2	7.7		128.9		71	33	7.5		127.6								
Mean	1	1	8.7		133.4		61	21	8.6		131.2								
		2	7.5		132.0		74	36	7.3		131.5								
790	2	1	8.4		140.2		85	31	8.0		132.3								
		2	7.9		133.7		100	47	7.7		135.2								
800	2	1	9.0		138.6		79	26	8.8		133.4								
		2	7.6		132.1		86	41	7.4		135.1								
816	2	1	9.1		135.1		62	22	8.5		132.4								
		2	7.7		133.7		73	33	7.4		133.2								
Mean	2	1	8.8		137.9		75	26	8.4		132.7								
		2	7.7		133.1		86	40	7.5		134.5								
Mean	1& 2	1	8.8		135.7		68	24	8.5		131.9								
		2	7.6		132.6		80	38	7.4		133.0								

[1] Time 1 = pre-performance traffic; Time 2 = post-performance traffic.
[2] 30-in. plate.

applications required to reduce serviceability to 1.5, was limited to 3 of the 13 pairs of sections (787-788, 793-794 and 811-812). Two of these instances indicated that those sections subjected to the conventional tires suffered a more rapid loss of serviceability; while the third instance showed a reversal of this effect (see Table 12).

Furthermore, comparison of the serviceability indexes for the other sections at the beginning and end of the special traffic clearly shows that a greater loss of serviceability occurred for those sections subjected to the conventional tires. The increase in serviceability, apparent in sections 815 and 816, was within the normal error for the serviceability index.

The increases in lineal feet of all classes of cracking for the rigid sections are given in Table 14. Figure 22 shows typical crack patterns. In general, at a given number of applications, the greater increases in length of cracks are associated with those sections subjected to the vehicles equipped with the LPLS tires. With only one exception the sections showing a greater increase in cracking are in lane 1 (LPLS tires). However, it may be noted that the amount of cracking prior to the start of the test was generally less in these sections than the amount of cracking in the comparable sections in lane 2.

The significance of the original condition of the sections becomes more important when it is recalled that all classes of cracking, 1 through 4, are combined in Table 14. This suggests that class 1 and/or class 1 and 2 cracking occurred under fewer applications, but failed to develop into class 3 or 4 (the only classes that deduct from the serviceability index).

The extent of edge pumping of the rigid pavements was investigated in an attempt to establish a basis for comparison of the effect of the two tire designs. Table 15 expresses the amount of pumping in terms of pumping score units for three levels of applications. The pumping score is determined by the following formula:

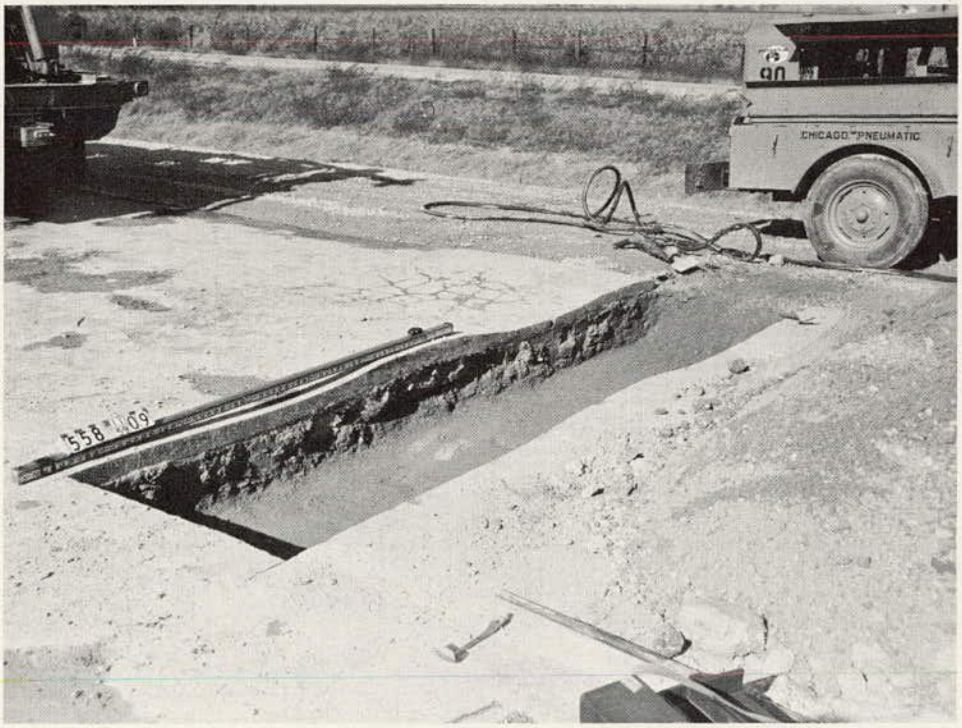


Figure 23. Pre-traffic trench for materials investigation.

Pumping Score = Percent length Trace + 10 (percent length Moderate) + 50 (percent length Heavy)

where Trace, Moderate and Heavy are terms indicative of the cross-sectional area of the pile of ejected material.

In all sections subjected to the 32-kip tandem axle load on conventional tires, the pumping score is greater than for the companion section subjected to the same loading with LPLS tires. However, it must be pointed out that the pumping score for these sections (lane 2) at the start of traffic was much larger than for sections in lane 1 (LPLS) where no pumping had been observed. Pumping continued to accelerate under the 32-kip tandem loads more rapidly in lane 2 (conventional tires) than in lane 1 (LPLS tires), as was expected.

The replicate sections (799-780, 815-816), subjected to the reverse traffic pattern, cannot be compared on the basis of serviceability loss. No severe distress contributing to loss of serviceability occurred in any of the sections under the 16,500 axle load applications. The relationships for these sections on the basis of increase in pumping score and cracking are given in Table 16.

TABLE 21

MATERIALS EDGE SAMPLING PROGRAM

(a) FLEXIBLE TANGENT

Section	Lane	Time[1]	Moisture Content (%) Base Course	Subbase	Embankment 0–1 In.	Embankment 0–4 In.
719	1	1	4.1	5.4	16.1	16.7
		2	4.6	5.6	15.3	16.8
		3	3.8	5.0	16.1	16.7
		4	4.1	5.4	15.7	16.7
741	1	1	4.0	5.3	15.1	17.3
		2	4.3	5.5	15.8	16.8
		3	3.6	4.6	15.8	16.5
		4	3.9	5.1	14.7	16.3
773	1	1	4.4	—	15.4	16.9
		2	4.4	—	16.0	17.0
		3	4.3	—	14.9	17.0
		4	4.1	—	16.2	16.1
Mean	1	1	4.2	5.3	15.5	17.0
		2	4.4	5.5	15.7	16.9
		3	3.9	4.8	15.6	16.7
		4	4.0	5.3	15.5	16.4
720	2	1	4.1	5.1	15.6	16.8
		2	4.7	5.6	16.1	17.1
		3	4.1	4.9	16.0	16.0
		4	4.4	5.4	16.1	17.1
742	2	1	4.4	5.1	15.4	16.7
		2	4.5	5.2	16.0	16.4
		3	3.9	4.6	14.8	16.2
		4	4.5	5.1	15.8	15.8
774	2	1	4.7	—	14.8	15.6
		2	5.1	—	16.5	16.5
		3	4.6	—	16.4	16.4
		4	3.9	—	15.8	17.5
Mean	2	1	4.4	5.1	15.3	16.4
		2	4.8	5.4	16.2	16.7
		3	4.2	4.8	15.7	16.2
		4	4.3	5.3	15.9	16.8
Mean	1 & 2	1	4.3	5.2	15.4	16.7
		2	4.6	5.5	16.0	16.8
		3	4.1	4.8	15.7	16.5
		4	4.2	5.3	15.7	16.6

(b) RIGID TANGENT

Section	Lane	Time[1]	Moisture Content (%) Base Course	Subbase	Embankment 0–1 In.	Embankment 0–4 In.
789	1	1	—	5.9	16.5	16.8
		2	—	5.7	15.9	16.6
		3	—	5.2	16.2	17.2
		4	—	5.4	15.3	16.5
799	1	1	—	5.6	16.3	16.6
		2	—	5.2	16.4	16.9
		3	—	5.5	16.6	17.3
		4	—	4.4	15.8	16.8
Mean	1	1	—	5.7	16.4	16.7
		2	—	5.4	16.1	16.7
		3	—	5.3	16.4	17.2
		4	—	4.9	15.6	16.6
790	2	1	—	4.9	15.6	16.9
		2	—	5.5	16.1	16.7
		3	—	5.1	15.9	16.7
		4	—	4.8	14.8	16.3
800	2	1	—	5.5	16.7	18.2
		2	—	5.3	16.3	17.7
		3	—	5.6	18.2	17.9
		4	—	5.7	16.3	17.6
Mean	2	1	—	5.2	16.2	17.5
		2	—	5.4	16.2	17.2
		3	—	5.4	17.0	17.3
		4	—	5.2	15.5	17.0
Mean	1 & 2	1	—	5.5	16.3	17.1
		2	—	5.4	16.2	17.0
		3	—	5.4	16.7	17.3
		4	—	5.1	15.6	16.8

[1] Dates sample: 1 = 4–5–61, 2 = 4–20–61, 3 = 5–5–61, 4 = 5–19–61.

TABLE 22

TRENCHES IN FAILED AREAS, FLEXIBLE

Section	Lane	Date	Inner Wheelpath: Moist. Cont. (%) 0–1 In.	Inner Wheelpath: Moist. Cont. (%) 0–4 In.	Inner Wheelpath: Dry Density (pcf)	Outer Wheelpath: Moist. Cont. (%) 0–1 In.	Outer Wheelpath: Moist. Cont. (%) 0–4 In.	Outer Wheelpath: Dry Density (pcf)	Outer Wheelpath: CBR (0.1 in.)	Outer Wheelpath: Plate Load Test (lb/cu in.) k_E	Outer Wheelpath: Plate Load Test (lb/cu in.) k_G	Between Wheelpaths: Moisture Content (%) 0–1 In.	Between Wheelpaths: Moisture Content (%) 0–4 In.	Between Wheelpaths: Moisture Content (%) 16 In.	Between Wheelpaths: Moisture Content (%) 30 In.	Between Wheelpaths: Dry Density (pcf) 0–1 In.	Between Wheelpaths: Dry Density (pcf) 16 In.	Between Wheelpaths: Dry Density (pcf) 30 In.	Between Wheelpaths: CBR (0.1 in.)
(a) EMBANKMENT																			
712	2	4/25/61	16.5	17.2	106.2	16.2	17.7	109.6				16.5	17.2	17.0	14.6	105.4	109.4	113.2	
737	1	4/25/61	16.2	16.8	108.6	17.2	17.2	110.7				16.2	16.8	17.0	15.5	107.7	106.4	114.2	
738	2	4/21/61	14.6	15.5	111.6	15.9	16.4	109.0				16.0	16.4	19.6	15.1	109.9	106.2	114.5	
758	2	4/ 4/61	15.4	18.0	106.2	14.4	16.8	107.6				14.4	18.5	15.5	15.5	107.5	111.0	114.6	
759	1	4/ 4/61	15.2	16.9	106.6	15.8	16.1	107.0				15.6	17.4	19.0	13.6	105.8	106.6	—	
763	1	6/ 6/61				13.8	15.1	113.1	3.0	101	71	14.4	15.6	16.5	13.2	111.2	110.8	116.5	2.9
764	2	6/ 5/61				14.8	16.8	109.6	3.3	112	66	14.6	15.9	14.6	12.7	109.0	114.2	116.8	2.7
Mean[1]	1 & 2	1[2]				15.5	17.1	109.2	1.8	90	42	15.7	16.8	15.9	14.6	108.6	108.6	113.5	1.7
		2[2]				15.1	16.4	109.4	2.8	94	56	15.2	16.3	16.6	14.9	109.1	108.3	112.8	2.4
(b) BASE COURSE																			
712	2	4/25/61	4.8			5.2													
737	1	4/25/61	4.6			4.6													
738	2	4/21/61	4.3			4.2													
758	2	4/ 4/61	4.3			4.1													
759	1	4/ 4/61	4.3			4.5													
763	1	6/ 6/61				4.1		140.8	50			4.3				140.2			69
764	2	6/ 5/61				4.1		148.4	56			4.1				147.4			69
Mean[1]	1 & 2	1[2]				4.6		137.4	31			4.7				137.8			35
		2[2]				4.0		140.2	55			4.0				137.5			46
(c) SUBBASE																			
712	2	4/25/61	6.0			5.9													
737	1	4/25/61	6.0			5.6													
738	2	4/21/61	5.6			5.7													
758	2	4/ 4/61																	
759	1	4/ 4/61																	
763	1	6/ 6/61				5.0		138.8	25			6.8				133.5			14
764	2	6/ 5/61				5.2		133.4	8			5.7				140.2			11
Mean[1]	1 & 2	1[2]				5.8		139.3	9.9			5.9				139.4			9.1
		2[2]				5.1		137.8	16.8			5.5				137.2			12.8

Note: In (b) and (c), a single value spans both moisture-content columns of the inner and outer wheelpaths, all four moisture-content columns between the wheelpaths, and all three dry-density columns between the wheelpaths.

[1] From Table 14.
[2] 1 = pre-performance traffic; 2 = post-performance traffic.

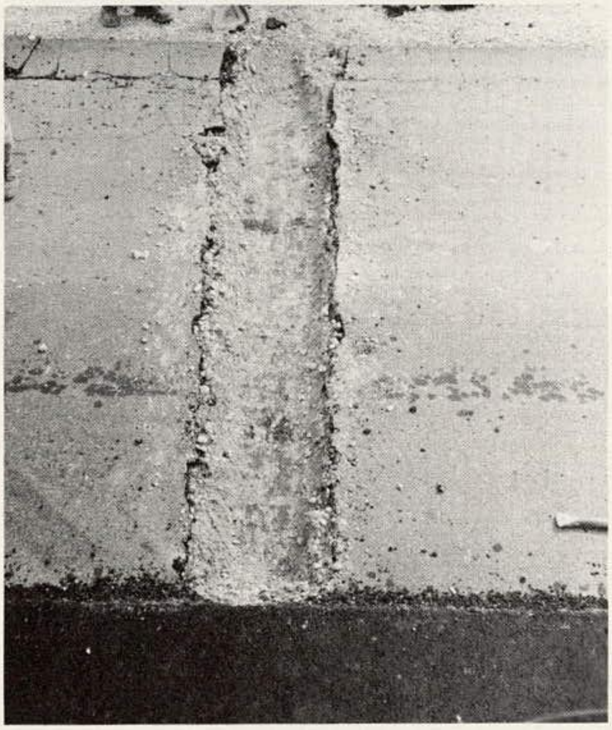


Figure 24. Trench adjacent to failed area.

TABLE 23

SECTIONS THAT HAD BEEN OVERLAID

Section	Lane	Original Design	Overlay Thickness (in.)	Serviceability Prior to Traffic	Tire Design
		(a) FLEXIBLE			
709	1	2–3–4	3.0	3.3	LPLS
710	2	2–3–4	3.0	2.3	Conv.
729	1	2–0–4	4.5	3.0	LPLS
731	1	2–3–0	4.5	3.7	LPLS
735	1	0–6–0	4.5	3.7	LPLS
739	1	3–0–4	3.0	3.6	LPLS
741	1	2–3–4	3.0	3.6	LPLS
742	2	2–3–4	4.5	3.0	Conv.
747	1	0–6–0	4.5	2.9	LPLS
756	2	1–6–0	6.0	2.5	Conv.
760	2	2–3–0	4.5	2.9	Conv.
765	1	0–3–4	4.5	3.5	LPLS
769	1	3–0–0	3.0	3.7	LPLS
773	1	3–3–0	3.0	2.8	LPLS
774	2	3–3–0	4.5	2.4	Conv.
		(b) RIGID			
781	1	2.5R–0	4.5	3.6	LPLS
786	2	2.5–6	4.5	3.9	Conv.
790	2	2.5–6	4.5	3.7	Conv.
805	1	2.5R–0	4.5	3.2	LPLS

2.6 MATERIALS INVESTIGATIONS

Measurements of the physical properties of the materials in the various structural layers of the test pavements were divided into three separate studies. First, trenches were opened in selected sections in both tangents prior to the pre-traffic maintenance program and again following the special traffic; second, samples of the embankment material were taken from the shoulders of several sections during the traffic program; and third, limited investigations were made adjacent to the failed areas.

The characteristics of the embankment soil are given in Table 17 and the characteristics of the subbase, base and asphaltic concrete are given in Table 18.

The data obtained in the trench program before and after traffic are shown in Tables 19 and 20 for both the flexible and rigid tangents. Figure 23 shows the trenching operation. From these data differences were found between the lanes (LPLS and conventional) which might help to explain the results reported in the foregoing sections.

The second series of subsurface studies was conducted at four times during the traffic period. Samples were taken in core holes made at the pavement edge in ten sections. The resulting data are shown in Table 21. The differences between lanes parallel the differences shown in Tables 19 and 20.

The final part of the materials investigation for this study was concerned with limited measurements of the properties of the materials in the structural layers immediately after failure of a section. This study was conducted in flexible sections only. Table 22 summarizes the data taken and includes, for comparison, the lane and tangent means determined from the trench program. Figure 24 shows a trench opened in one of the failed sections.

The data shown for sections 763 and 764 are more complete than for other sections. These sections were of the heaviest design in the tangent and, although seriously distressed, had not been maintained prior to this investigation. A review of these data failed to show any differences between lanes at the time of failure.

2.7 OVERLAY STUDIES

Table 23 lists all sections, both rigid and flexible, that had been overlaid either during the regular traffic period or just before performance study. It gives their original design and the thickness of the asphaltic concrete overlay.

TABLE 24

OBSERVATIONS ON OVERLAID FLEXIBLE SECTIONS

Section	Orginal Design (in.)	Overlay (in.)	Tire Design	Item[1]	Values at Applications Indicated: 0	338	854	962	1,636	2,874	3,494	4,082	5,046	6,284	8,514	10,456	14,230	16,500
709	2–3–4	3	Conv.	(1)	0.0	0.0	0.0	0.0	0.02									
				(2)	0.0	0.0	0.0	0.0	0.0									
				(3)	3.3	3.3	3.4	3.0	3.1									
710	2–3–4	3	LPLS	(1)	0.0	0.0	0.0	0.0	0.02	0.05	0.04							
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	11							
				(3)	2.3	3.3	3.3	3.4	3.4	3.5	2.9							
729	2–9–4	4.5	LPLS	(1)	0.04	0.0	0.0	0.0	0.05	0.0	0.07							
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0							
				(3)	3.0	3.5	3.7	2.4	2.1	3.5	1.8							
731	2–3–0	4.5	LPLS	(1)	0.04	0.0	0.02	0.0	0.05	0.04	0.05	0.05	0.11	0.08	0.15	0.22	0.22	0.30
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
				(3)	3.7	3.6	3.8	3.7	3.4	3.8	4.0	3.3	3.8	3.4	3.9	3.8	3.7	3.5
735	0–6–0	4.5	LPLS	(1)	0.04	0.02	0.0	0.0	0.05	0.02	0.04	0.07	0.09	0.05	0.12	0.12	0.12	0.16
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
				(3)	3.7	3.6	3.8	3.7	3.7	3.6	3.7	3.5	3.5	3.8	3.4	3.4	3.7	3.6
739	3–0–4	3	LPLS	(1)	0.0	0.02	0.0	0.02	0.05	0.09	0.20							
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0							
				(3)	3.6	3.4	3.5	3.5	3.4	3.7	3.0							
741	2–3–4	3	LPLS	(1)	0.0	0.0	0.0	0.0	0.05	0.02	0.17	0.15	0.17	0.14	0.23	0.30	0.32	0.38
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
				(3)	3.6	3.3	3.5	3.5	3.6	3.4	3.4	3.1	3.0	2.8	2.9	2.7	2.4	2.6
742	2–3–4	4.5	Conv.	(1)	0.0	0.02	0.02	0.05	0.07	0.04	0.04	0.09	0.05	0.09				
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0				
				(3)	3.0	2.9	3.0	3.0	2.9	3.1	2.9	2.8	2.5	2.2				
747	0–6–0	4.5	LPLS	(1)	0.0	0.03	0.02	0.04	0.09									
				(2)	0.0	0.0	0.0	0.0	5									
				(3)	2.9	3.3	3.2	3.1	3.4									
756	1–6–0	6	Conv.	(1)	0.07	0.02	0.03	—	0 03									
				(2)	0.0	0.0	0.0	0.0	0.0									
				(3)	2.5	2.7	2.6	—	2.5									
760	2–3–0	4.5	Conv.	(1)	0.10	0.05	0.05	—	0.04									
				(2)	0.0	0.0	0.0	0.0	0.0									
				(3)	2.9	2.6	2.7	—	2.7									
765	0–3–4	4.5	LPLS	(1)	0.0	0.04	0.02	0.05	0.10	0.05	0.09	0.12	0.13	0.12	0.17	0.17	0.20	
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	8	21	
				(3)	3.5	3.5	3.5	3.4	3.5	3.5	3.3	3.4	3.7	3.3	3.3	3.0	2.1	
769	3–0–0	3	LPLS	(1)	0.0	0.04	0.0	0.0	0.05	0.04	0.10							
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	100							
				(3)	3.7	3.5	3.7	3.4	3.6	3.6	3.4							
773	3–3–0	3	LPLS	(1)	0.0	0.0	0.0	0.0	0.02									
				(2)	0.0	0.0	0.0	0.0	0.0									
				(3)	2.8	3.0	2.9	2.7	2.6									
774	3–3–0	4.5	Conv.	(1)	0.0	0.02	0.03	0.05	0.07									
				(2)	0.0	0.0	0.0	0.0	0.0									
				(3)	2.4	2.6	2.4	2.3	1.9									

[1] (1) = Rut depth (in).; (2) = Cracking (lin ft); (3) = Serviceability.

TABLE 25

OBSERVATIONS ON OVERLAID RIGID SECTIONS

Section	Original Design (in.)	Overlay (in.)	Tire Design	Item[1]	Applications 0	338	854	962	1,636	2,874	3,494	4,082	5,046	6,284	8,514	10,456	14,230	16,500
781	2.5–0	4.5	LPLS	(1)	0.0	0.01	0.01	0.02	0.04	0.01	0.03	0.04	0.05	0.05	0.06	0.08	0.08	0.11
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
				(3)	—	3.7	3.7	3.5	3.4	3.8	3.5	3.5	3.5	3.5	3.6	3.6	3.6	3.6
805	2.5–0	4.5	LPLS	(1)	0.0	0.0	0.0	0.0	0.03	0.01	0.02	0.04	0.07	0.07	0.08	0.12		
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	2.4		
				(3)	3.1	3.3	3.3	3.2	3.4	3.5	3.4	3.3	3.3	3.3	3.3	3.2		
786	2.5–6	4.5	Conv.	(1)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.03	0.04	0.06	0.09	0.10	0.10
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
				(3)	3.9	3.8	3.6	3.6	3.7	3.1	3.2	3.2	3.1	3.2	3.2	3.3	3.3	3.1
790	2.5–6	4.5	Conv.	(1)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.03	0.02	0.04	0.06	0.07	0.09
				(2)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
				(3)	3.7	3.8	3.0	3.9	3.9	3.9	—	4.0	3.8	3.8	3.9	3.8	3.8	3.8

[1] (1) = Rut depth (in.); (2) = Cracking (lin ft); (3) = Serviceability.

Considerable difficulty was encountered in the construction of satisfactory overlays as short as one flexible pavement test section (100 ft). Observation of the performance of these beyond a few thousand applications was difficult since the ramps up and down from the full thickness of overlay showed distress, in the form of shoving and cracking, soon after traffic started. This necessitated the addition of more surfacing material, which produced a major change in the serviceability index. For these reasons analysis of the performance of the flexible sections that had been overlaid was not practical.

The overlaid rigid sections were longer (120 and 240 ft) and thus the ramp effect was not critical. They performed satisfactorily with a slight loss in serviceability associated with the development of rutting in the overlay of about 0.1 in. In only one instance (section 805) did the ramp effect necessitate discontinuing the observation of the section.

Tables 24 and 25 give the mean rut depth, lineal feet or area of cracking (and patching for the flexible sections) and the serviceability for the flexible and rigid sections listed in Table 23.

2.8 NEEDED RESEARCH

Although some trends were shown in this study, it was clear that additional research is needed before the relative effects on pavement performance of vehicles equipped with the two types of tire can be evaluated.

The additional work may well take the form of that reported here except that the different tire designs and pressures should be operated over test pavements that are either new or that have identical traffic histories. Provisions should be made for a much greater number of test load applications and for more instrument installations for obtaining measurements relating to performance or serviceability than were available at the time of this study at the AASHO Road Test.

Additional research is also needed in which the mechanics of pavement behavior under load is studied. Theoretical studies of layered systems should be encouraged and field tests of the theories should follow.

Chapter 3

Tire Pressure—Tire Design

3.1 SUMMARY

The objective of this program was to investigate the possible effect of changes in tire pressure and design on the dynamic measurements associated with pavement structures and bridges. Instrumentation was available on the Road Test to measure dynamic strains and deflections, dynamic loads, transmitted embankment pressures and bridge responses.

The tests were conducted on pavement sections in Loops 4 and 6 which had survived over 1 million applications of either 18-kip or 30-kip single axle loads and on bridges that had been subjected to over 500,000 vehicle passages.

Pilot studies conducted previously on the Road Test helped to define the range of vehicles and loads tested; and the findings of these studies predicted the findings of the studies described in this chapter.

Flexible Pavement Study

Generally, tire inflation pressure changes of the order of 40 to 50 psi accompanied by a limited tire design change had little effect on the dynamic deflections of the pavement sections included in this study.

Relationships similar to those found in the side studies of the main AASHO Road Test (Report 5) existed between wheel load and deflection, indicating that the data accumulated were rational.

Changes in tire pressure or design had little or no effect on the pressure transmitted to the embankment. Such changes may, or may not, affect transmitted pressures within the pavement structure. However, no instruments were available to measure such phenomena.

The area of maximum transmitted pressure appeared to increase with increase in tire inflation pressure which is normally associated with a decrease in tire contact area.

Rigid Pavement Study

Changes in tire pressure or tire design produced no noticeable effect on the dynamic edge strains or deflections. However, had instrumentation been available to measure strain or deflection in the pavement surface at points other than at the edge, effects may have been found.

The relationships found between wheel load and deflection and strain and between vehicle speed and deflection and strain agree with those presented in AASHO Road Test Report 5.

Dynamic Load Study

The dynamic load effect was clearly related to vehicle speed and pavement serviceability. However, the effects of changes in tire pressure or design were not consistent within the range included in this study.

Bridge Study

The possible effects of the different tire pressures and designs on the bridge responses must be considered only as trends since the characteristics of the vehicles and the vehicle load patterns were not uniform.

In general, the strain and deflection amplification factor (dynamic strain or deflection as a ratio of static strain or deflection) relationships found in this study agree with those given in AASHO Road Test Report 4, that is, an increase in speed was associated with larger amplification factors and the amplification factors for single axle vehicles were generally greater than those for the tandem axle vehicles.

3.2 SCOPE

To evaluate the effect of different loads and tire pressures-tire designs on the pavements, bridges and cargoes, a program was conducted including measurements of dynamic strains and deflections, pressures transmitted to embankment soil, accelerations in the vehicle and tire pressure-dynamic load relationships. In this chapter, all these relationships are discussed except the vehicle and cargo accelerations which are grouped for all vehicles and studies in Chapter 8.

3.3 DESCRIPTION OF MEASUREMENTS

Eighteen vehicles (including two replicates) were divided into four categories with 18-kip and 22.4-kip single axle and 32-kip and 40-kip tandem axle loads. In each of the load groups, there were vehicles equipped with wire and nylon cord tires at several sizes and inflation pressures. Table 26 lists the vehicles and certain of their characteristics.

These vehicles grouped as a train were operated over pavement sections in Loop 6 equipped with electronic devices for measuring deflection and strain, and in Loop 4 equipped with electronic devices for measuring embankment soil pressures, and over bridge structures of different designs on Loop 6 equipped with devices to measure strain and midspan deflection. Figure 25 is a schematic

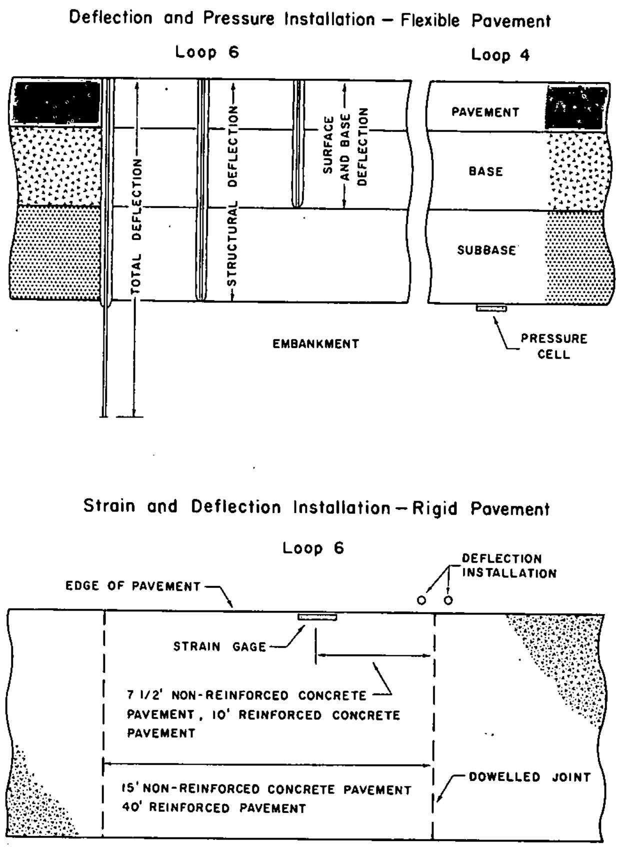


Figure 25. Typical instrument installation.

drawing of the pavement instrument installations for both Loops 4 and 6.

Four sections each of flexible and rigid pavement were included in the study. The bridge structures included had not yet been subjected to the dynamic overload studies described in Road Test Report 4. Table 27 gives the pavement sections and structures tested, and notations of the measurements that were taken.

The selection of the sections for this study was limited to the thickest designs on the Road Test. Furthermore, sections which had experienced very little or no distress during the regular test were selected to insure uniformity.

The train of vehicles was operated over each of the sections and bridges in random order at three levels of speed (creep, 15 and 30 mph). At least three passes within a prescribed transverse placement were required for each vehicle at each speed level. The tests were conducted, as far as practicable, in only one flexible and one rigid section per day to minimize the temperature effect for those sections and to make within section studies possible. However, a section-to-section analysis (either rigid or flexible) was not feasible because of wide temperature fluctuations during the study.

In addition to the studies of dynamic strain and deflection, a study was conducted of the dynamic load applied to the pavement. The measuring device and the data from this study are discussed subsequently in this chapter.

The data accumulated in the tire pressure-

TABLE 26

VEHICLE CHARACTERISTICS

Axle Load (kips)	Tire Size [1]	Tire Pressure (psi)	Gross Load (kips)	Axle Load (kips)					Axle Spacing (in.)				Center of Duals (in.)	Gross Tire Contact Area [2] (sq in.)
				1	2	3	4	5	1–2	2–3	3–4	4–5		
18 S	9.00x20N	100	29.8	4.3	7.5	18.0			144	246.5			72	254.0
18 S	10.00x20N	80	30.0	4.6	7.2	18.2			144	246			72	274.0
18 S	10.00x20N	80	40.3	5.6	16.9	17.8			143	246.5			72	296.4
18 S	12.00x20N	60	30.3	4.7	7.4	18.2			143	246.5			72	335.2
18 S	8.25x20W	115	30.3	4.7	7.4	18.2			144	246.5			72	220.0
22.4 S	10.00x20N	100	30.8	4.9	6.7	22.2			137	246			72	284.8
22.4 S	11.00x20N	80	34.4	5.2	6.9	22.3			137	246			72	256.8
22.4 S	12.00x20N	60	34.0	5.1	6.6	22.3			137	246.5			71	368.0
22.4 S	9.00x20W	125	34.0	5.3	6.6	22.1			137	246.5			71	244.4
32 T	8.25x20N	100	52.2	8.3	6.1	5.9	13.3	18.6	128	48	244	48	72	370.4
32 T	9.00x20N	80	52.3	8.2	6.1	5.8	14.4	17.8	129	48	244	50	72	469.6
32 T	11.00x20N	60	53.4	8.4	5.9	5.9	16.6	16.6	128	48	237	50	72	462.4
32 T	8.25x20W	100	51.5	8.3	5.9	5.7	14.0	17.6	128	48	241	50	72	381.6
40 T	9.00x20N	100	62.4	7.8	7.6	7.3	18.6	21.1	144	50	241	50	72	479.2
40 T	11.00x20N	80	62.5	7.8	7.7	7.2	17.8	22.0	144	52	240	50	72	552.0
40 T	11.00x20N	80	90.3	9.0	20.4	20.1	20.7	20.1	144	50	241	50	72	552.0
40 T	12.00x20N	60	66.3	9.9	8.4	7.9	19.4	20.7	132	54	239	50	72	630.4
40 T	8.25x20W	130	65.8	8.3	9.2	8.9	19.8	19.6	143	52	240	50	72	447.2

[1] N = nylon cord; W = wire cord.
[2] Per axle or axles.

tire design study are presented and discussed as they were taken in the individual studies: first, the relationships for the flexible sections; second, the relationships for the rigid sections; third, the discussion of the tire pressure-dynamic load relationships; and fourth, the relationships for the bridge structures.

3.4 FLEXIBLE PAVEMENT STUDY

During the planning stage of the special studies, two pilot studies were conducted to investigate the effect of the change of tire pressure and/or design. The first was in October 1959 and involved a comparison of conventional tire designs at two levels of inflation pressure (40 and 70 psi). The second was conducted in June 1960. At this time, the LPLS tires were available, and the study included as variables both tire design (LPLS and conventional) and tire pressure (23 to 70 psi).

The findings reported in these limited studies helped to design the experiment discussed here. In the first pilot study, no significant differences were noted in the values for dynamic strain or deflection for either the flexible or rigid sections. In the second pilot study, it was found that when a pressure change from 23 to 70 psi was accompanied by a tire design change, there appeared to be a significant difference in the values of both strain and deflection. The low pressure tires caused slightly greater strains and deflection than did the higher pressure tires.

The tire pressure-tire design study for the flexible sections was based to a considerable extent on these pilot tests. Table 26 lists the range of tire-pressures and tire designs incorporated in the study, and Table 27 lists the pavement sections and structures tested.

Table 28 gives the mean total deflections under each loading for the four flexible sections included in the study. The embankment and structural deflections for these sections indicated that similar tire-deflection relationships existed at all levels. Each value is the mean of at least four field readings at a controlled transverse placement. The temperature of the asphaltic concrete surfacing during the tests in each section varied less than 10 deg. The range of temperature for all days required to complete the study of the four sections was 47 to 80 F. Normal relationships between deflection and wheel load and between deflection and vehicle speed existed (Table 28). The data are further reduced in Table 29.

The findings from this portion of the tire pressure-tire design study compare favorably

TABLE 27

INSTRUMENTED SECTIONS AND STRUCTURES

Loop	Section	Tangent	Thickness (in.)			Measurement
			Surface	Base	Subbase	
			(*a*) SECTIONS			
6	349	Rigid	12.5		6	Strain and deflection
6	359	Rigid	12.5 R		3	Strain and deflection
6	367	Rigid	9.5		6	Strain and deflection
6	381	Rigid	9.5 R		3	Strain and deflection
6	389	Rigid	9.5		6	Strain and deflection
6	397	Rigid	11		6	Strain and deflection
6	265	Flexible	5	9	16	Deflection
6	271	Flexible	6	9	8	Deflection
6	301	Flexible	6	6	16	Deflection
6	333	Flexible	6	9	16	Deflection
4	581	Flexible	5	6	12	Embankment pressures
			(*b*) BRIDGES			
6	3–B	Steel composite, 27 ksi, cover plate 18 WF 60				Strain and deflection
6	8–A	Reinforced conc. monolithic, 30 ksi, 3 No. 11, 2 No. 1, 1 No. 8				Strain and deflection
6	8–B	Reinforced conc. monolithic, 30 ksi, 3 No. 11, 2 No. 9, 1 No. 8				Strain and deflection
6	9–A	Steel composite, 27 ksi, cover plate 18 WF 60				Strain and deflection
6	9–B	Steel noncomposite, 27 ksi, cover plates 18 WF 96				Strain and deflection

TABLE 28

Total Deflection Values for Flexible Sections

(a) Section 265, 5–9–16 Design — Total Deflection (in. × 10^{-3})

Axle Load (kips)	Tire Press. (psi)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	Mean OWP	Mean IWP	Mean Sec.
18 S	100	18	14	15	16	13	12	16	13	15
	80	16	15	14	15	15	13	15	14	14
	80	18	17	14	16	14	13	16	14	15
	60	18	15	14	14	12	10	16	12	14
	115W	18	16	14	16	12	12	16	13	15
22.4S	100	21	18	17	18	15	14	19	16	17
	80	22	19	15	20	17	14	19	17	18
	60	22	18	17	20	17	16	19	18	18
	125W	21	16	17	19	15	15	18	16	17
32 T	100	17	14	11	13	11	10	14	11	13
	80	15	13	12	15	13	12	13	13	13
	60	18	16	13	13	11	9	16	11	14
	100W	16	13	12	12	10	10	14	11	12
40 T	100	20	16	15	16	14	12	17	14	15
	80	19	16	14	15	13	11	16	13	15
	80	22	18	15	18	15	13	18	15	17
	60	20	19	16	18	15	14	18	16	17
	130W	19	16	13	18	15	13	16	15	16

(b) Section 271, 6–9–8 Design — Total Deflection (in. × 10^{-3})

Axle Load (kips)	Tire Press. (psi)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	Mean OWP	Mean IWP	Mean Sec.
18 S	100	29	27	25	27	21	17	27	22	24
	80	31	31	26	26	27	17	29	23	26
	80	34	29	25	27	22	19	29	23	26
	60	33	27	23	25	18	16	28	20	24
	115W	33	28	22	26	21	19	28	22	25
22.4S	100	39	35	—	34	27	—	37	30	34
	80	39	35	29	31	28	18	31	25	28
	60	39	32	29	33	24	18	33	25	29
	125W	37	33	26	34	28	20	32	27	30
32 T	100	28	23	21	22	17	15	24	18	21
	80	27	23	21	26	21	17	24	21	23
	60	31	29	25	23	19	16	28	19	24
	100W	28	25	29	22	19	15	27	19	23
40 T	100	35	30	24	28	22	17	30	22	26
	80	36	30	26	25	19	16	30	20	25
	80	39	35	28	32	27	19	34	26	30
	60	35	32	25	28	26	19	30	24	27
	130W	35	28	25	29	26	17	29	24	27

(c) Section 301, Design 6–6–16 — Total Deflection (in. × 10^{-3})

Axle Load (kips)	Tire Press. (psi)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	Mean OWP	Mean IWP	Mean Sec.
18 S	100	17	16	13	15	14	9	15	13	14
	80	15	14	11	15	14	9	13	13	13
	80	19	18	14	16	17	11	17	15	16
	60	16	14	13	15	11	10	14	12	13
	115W	17	16	14	17	13	11	16	14	15
22.4S	100	22	21	15	21	18	13	19	17	18
	80	32	20	11	30	17	10	21	19	20
	60	23	17	17	20	14	12	19	15	17
	125W	19	20	17	19	17	15	19	17	18
32 T	100	17	14	12	14	12	10	16	12	14
	80	14	14	11	15	13	11	13	13	13
	60	16	16	11	12	12	9	16	11	14
	100W	15	14	11	13	12	10	13	12	12
40 T	100	20	18	13	17	14	10	17	14	15
	80	18	18	12	15	15	10	16	13	15
	80	24	21	15	21	18	14	20	18	19
	60	25	20	12	18	16	12	19	15	17
	130W	19	16	13	18	16	13	16	16	16

(d) Section 333, Design 6–9–16 — Total Deflection (in. × 10^{-3})

Axle Load (kips)	Tire Press. (psi)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	Mean OWP	Mean IWP	Mean Sec.
18 S	100	24	24	18	15	15	10	22	13	18
	80	26	23	16	18	16	11	22	15	18
	80	27	24	18	16	15	12	23	14	18
	60	26	24	18	14	13	9	23	12	17
	115W	27	22	18	17	12	11	22	13	17
22.4S	100	30	27	22	19	18	12	26	16	21
	80	33	25	21	20	14	11	26	15	20
	60	29	24	22	18	14	12	25	15	20
	125W	29	25	20	18	14	11	25	14	20
32 T	100	21	19	15	12	10	8	18	10	14
	80	21	16	15	15	11	9	17	11	14
	60	23	20	17	11	9	8	20	9	14
	100W	21	19	17	12	10	8	19	10	14
40 T	100	26	25	20	15	14	11	24	13	18
	80	26	22	19	15	11	10	22	12	17
	80	29	25	20	18	15	12	25	15	20
	60	29	24	21	16	13	12	25	14	20
	130W	26	20	18	17	12	12	21	14	17

with the findings reported for the two pilot studies previously mentioned. Generally, tire inflation pressure changes of the order of 40 to 50 psi accompanied by limited tire design changes had little effect on the dynamic deflection of flexible pavements of the designs tested.

Embankment pressures were also measured in Loop 4 under the train of vehicles. The gage selected for this study was located in the inner wheel path of section 581 whose pavement design was 5-6-12. This section was selected because its serviceability was higher than the serviceabilities of other sections with pressure gage installations.

The values of transmitted pressure for the 18 vehicles are given in Table 30, which includes the maximum transmitted pressure for each loaded axle or wheel and the distance from the axle at which the analog recording indicated a zero pressure reading. In addition, the mean values of pressure and distance are computed.

Figure 26 shows the relationship of transmitted pressure to vehicle speed. Up to a maximum speed of 30 mph, there was a highly significant decrease in the transmitted pressure at the embankment level with increase in speed. There was, however, no consistent effect on the pressure transmitted to the embankment with change in tire pressure or tire design.

Figure 27 shows the relationship existing between the wheel load and transmitted pressure. A curvilinear relationship was apparent in this study with a maximum pressure at a wheel load of about 10 kips. No consistent effect of tire type or pressure was noted. No explanation was found for the apparent drop in pressure with increase in load above 10 kips.

The data in Table 30 also failed to establish a trend in the maximum values for transmitted pressure at the prescribed transverse placement with respect to inflation pressure. The variations among individual readings were greater than the difference between the mean pressure values reported for all vehicles.

The distance from the loaded wheel at which a zero reading of transmitted pressure was recorded appears to have been linearly related to the wheel load. For the same wheel load the effect of a tandem axle was identical to that of a single axle when the distance was measured from the closest axle of the set whether leading or trailing.

In addition to the study of maximum values, a transverse and longitudinal pressure distribution study was conducted. The longitudinal

TABLE 29

FLEXIBLE PAVEMENT TOTAL DEFLECTION, SECTION AVERAGES[1]

Tire Pressure (psi)	Deflection (in. × 10^{-3}) 18 S Axle Load	22.4 S Axle Load	32 T Axle Load	40 T Axle Load	Avg.	Deflection (in. × 10^{-3}) 18 S Axle Load	22.4 S Axle Load	32 T Axle Load	40 T Axle Load	Avg.
	(a) SECTION 265, 5–9–16 DESIGN					(b) SECTION 271, 6–9–8 DESIGN				
60	14	18	14	17	15.8	24	29	24	27	26.0
80	14	18	13	16	15.3	26	28	23	27	26.0
100	15	17	13	15	15.0	24	34	21	26	26.3
Wire[2]	15	17	12	16	15.0	25	30	23	27	26.3
Avg.	14.5	17.5	13.0	16.0		24.8	30.3	22.8	26.8	
Mean range of each value[3] is 20 to 11.						Mean range of each value is 36 to 16.				
	(c) SECTION 301, 6–6–16 DESIGN					(d) SECTION 333, 6–9–16 DESIGN				
60	13	17	14	17	15.3	17	20	14	20	17.8
80	14	20	13	17	16.0	18	20	14	18	17.5
100	14	18	14	15	15.3	18	21	14	18	17.8
Wire[2]	15	18	12	16	15.3	17	20	14	17	17.0
Avg.	14.0	18.3	13.3	16.3		17.5	20.3	14.0	18.3	
Mean range of each value is 23 to 10.						Mean range of each value is 28 to 10.				

[1] Mean, both wheelpaths and all speeds.
[2] Wire cord tires at pressures 115, 125, 100 and 130 psi.
[3] Average of nine readings.

TABLE 30

TRANSMITTED EMBANKMENT PRESSURE VALUES, FLEXIBLE SECTION 581 (5–6–12 DESIGN)

Axle Load (kips)	Tire Size[1] (in.)	Tire Press. (psi)	Zero Reading[2] (in.)			Maximum Pressure (psi)						Zero Reading[3] (in.)			Mean Zero[2] Reading (in.)	Mean Max. Pressure (psi)		Mean Zero[3] Reading (in.)
						First Axle			Second Axle									
			Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph		First	Second.	
118 S	9.00x20N	100	46	45	38	5.37	5.05	5.13				56	52	52	43	5.18		53
	10.00x20N	80	40	46	48	5.30	5.00	3.85				50	55	62	44	4.72		56
	10.00x20N	80	44	50	48	4.52	5.20	4.53				54	65	58	47	4.75		59
	12.00x20N	60	44	49	48	5.30	4.70	4.55				52	60	62	47	4.85		58
	8.25x20W	115W	41	45	41	5.40	4.85	4.37				53	51	56	42	4.87		53
22.4 S	10.00x20N	100	50	54	52	6.00	4.70	3.40				58	54	68	52	4.70		60
	11.00x20N	80	48	63	62	6.40	4.97	3.53				55	67	78	58	4.97		66
	12.00x20N	60	53	51	48	5.75	4.80	3.40				60	60	64	51	4.65		61
	9.00x20W	125W	48	56	60	5.70	4.50	3.45				56	60	66	55	4.55		61
32 T	8.25x20N	100	41	48	44	3.85	3.25	2.80	5.15	4.20	3.15	52	56	66	44	3.30	4.17	58
	9.00x20N	80	42	52	50	4.10	3.35	2.40	5.40	3.90	3.75	49	65	60	48	3.28	4.35	58
	11.00x20N	60	39	29	48	4.35	3.70	2.70	4.10	3.53	2.13	42	60	63	39	3.58	3.25	55
	8.25x20W	100W	50	53	47	3.95	3.25	2.83	5.10	4.20	3.37	60	69	63	50	3.34	4.22	64
40 T	9.00x20N	100	45	52	40	5.85	4.57	4.90	7.05	5.53	5.60	47	68	52	46	5.11	6.06	56
	11.00x20N	80	42	60	40	5.53	4.20	3.95	6.63	5.20	4.70	51	66	54	47	4.56	5.51	57
	11.00x20N	80	49	54	43	6.00	5.07	4.83	6.20	5.10	4.97	56	65	55	49	5.30	5.42	58
	12.00x20N	60	46	45	40	6.05	5.50	4.60	6.25	5.60	4.30	55	60	52	44	5.38	5.38	56
	8.25x20W	130W	46	50	42	5.40	3.93	4.35	7.05	5.40	5.70	54	62	60	46	4.56	6.05	59

[1] N = nylon cord; W = wire cord.
[2] Leading distance to 0 pressure.
[3] Trailing distance to 0 pressure.

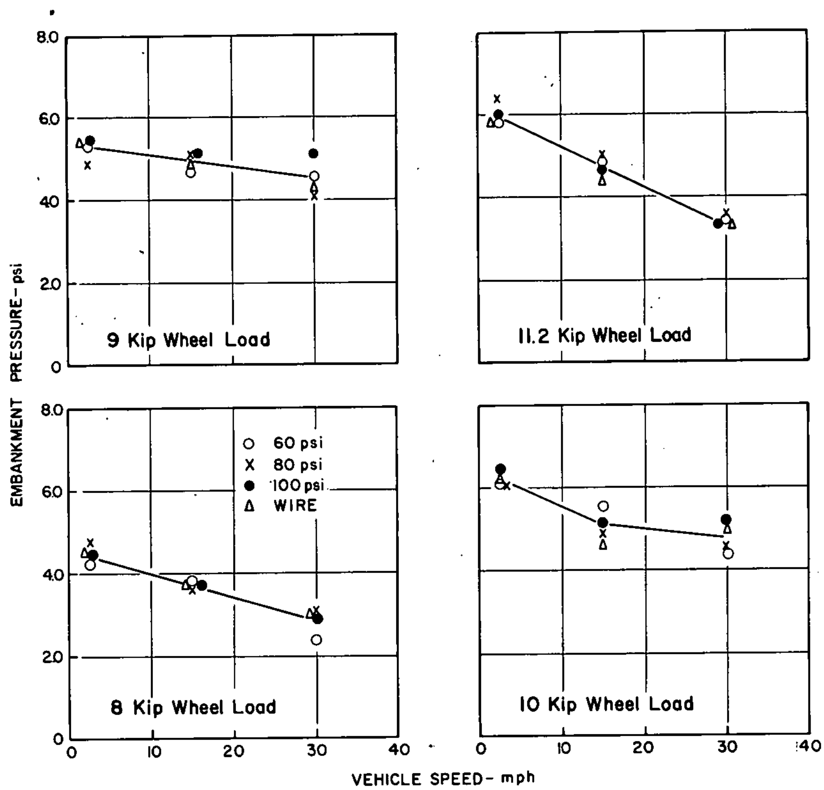


Figure 26. Relationship between vehicle speed and embankment pressure, Section 581 (5-6-12 design).

and transverse position of the loaded wheel was noted and the pressures at certain intervals were recorded. The influence curves shown in Figure 28 for the four tire designs and pressures for the 18-kip single axle loads are typical of those under all single-axle vehicles. These plots were developed from the data listed in Road Test Data System 9166.

In these four plots the influence area (to a 2.0-psi line) was, in general, the same for each of the different tire pressures. However, the area in which the transmitted pressure was greater than 5.0 psi was generally greater at the higher tire pressures (which are normally accompanied by a decrease of tire contact area).

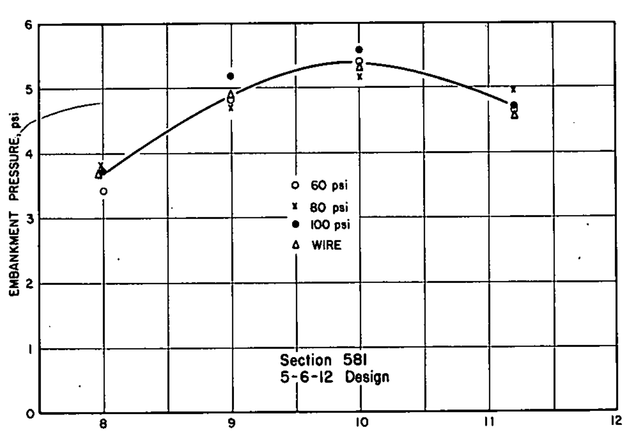


Figure 27. Relationship between wheel load and embankment pressure; mean values for creep, 15 and 30 mph.

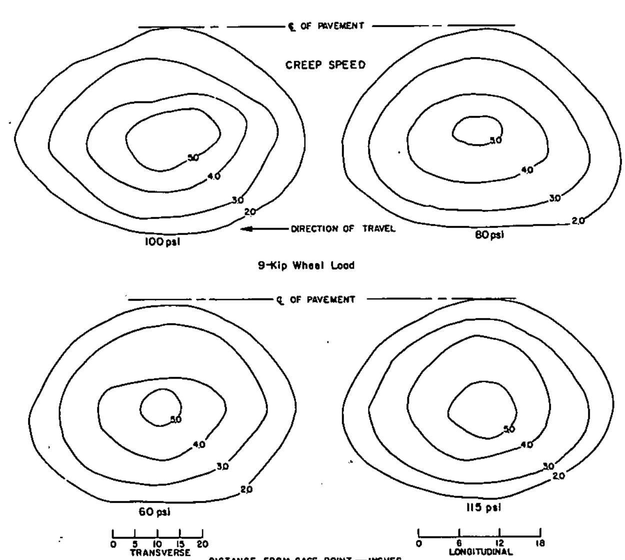


Figure 28. Embankment pressure influence diagrams, Section 581 (5-6-12 design).

TABLE 31

STRAIN AND DEFLECTION VALUES FOR RIGID SECTIONS

(a) SECTION 359, 12.5R–3 DESIGN

Axle Load (kips)	Tire Press. (psi)	Strain (10^{-6} in./in.) Compressive Creep	Compressive 15 Mph	Compressive 30 Mph	Tensile Creep	Tensile 15 Mph	Tensile 30 Mph	Deflection (10^{-3} in.) Creep	Deflection 15 Mph	Deflection 30 Mph
18 S	100	16	15	15	6	5	4	14	11	9
	80	16	15	13	5	5	6	14	11	11
	80	15	18	15	8	7	6	14	12	11
	60	18	16	16	5	6	4	14	11	9
	115W	15	15	14	6	6	5	14	12	9
22.4 S	100	19	19	17	7	6	6	16	13	12
	80	20	19	18	7	6	6	17	13	12
	60	20	21	16	6	7	7	15	15	13
	125W	19	17	18	7	7	5	16	14	13
32 T	100	16	16	12	8	7	6	19	17	12
	80	17	14	14	7	7	6	20	16	14
	60	15	14	12	8	7	6	21	16	13
	100W	17	17	14	7	8	6	20	17	13
40 T	100	21	21	18	9	9	8	24	20	18
	80	22	19	17	9	9	7	26	20	17
	80	17	18	16	13	12	10	25	21	18
	60	19	19	16	10	10	8	24	21	16
	130W	21	18	17	10	8	7	25	20	17

(b) SECTION 367, 9.5–6 DESIGN

Axle Load (kips)	Tire Press. (psi)	Strain (10^{-6} in./in.) Compressive Creep	Compressive 15 Mph	Compressive 30 Mph	Tensile Creep	Tensile 15 Mph	Tensile 30 Mph	Deflection (10^{-3} in.) Creep	Deflection 15 Mph	Deflection 30 Mph
18 S	100	24	24	24	12	12	10	26	25	20
	80	24	23	21	13	12	12	27	24	22
	80	26	22	26	15	15	11	30	28	17
	60	25	24	23	13	14	11	29	27	20
	115W	24	24	21	12	13	9	28	26	15
22.4 S	100	28	29	23	14	16	11	31	30	20
	80	30	29	31	15	15	10	33	31	18
	60	26	27	25	16	14	11	30	28	20
	125W	27	26	27	14	15	12	31	28	21
32 T	100	21	19	17	17	15	10	34	34	21
	80	21	21	17	15	15	13	36	30	26
	60	17	19	15	15	15	13	34	32	26
	100W	20	22	17	15	14	13	36	34	27
40 T	100	28	26	22	20	17	16	44	39	31
	80	30	28	22	18	17	15	41	34	32
	80	25	22	20	20	20	15	43	42	27
	60	24	22	23	19	17	13	41	39	25
	130W	26	25	23	19	18	11	42	36	26

(c) SECTION 381, 9.5R–3 DESIGN

Axle Load (kips)	Tire Press. (psi)	Strain (10^{-6} in./in.) Compressive Creep	Compressive 15 Mph	Compressive 30 Mph	Tensile Creep	Tensile 15 Mph	Tensile 30 Mph	Deflection (10^{-3} in.) Creep	Deflection 15 Mph	Deflection 30 Mph
18 S	100	21	20	20	6	7	7	14	12	14
	80	22	22	22	7	7	6	14	12	14
	80	22	24	16	8	8	8	14	14	11
	60	19	22	21	6	7	7	13	12	13
	115W	21	21	17	6	6	6	13	12	11
22.4 S	100	23	22	20	7	6	7	14	12	12
	80	25	26	23	7	7	7	15	15	13
	60	24	24	22	6	7	8	15	13	14
	125W	24	21	21	7	7	7	17	13	15
32 T	100	20	19	16	9	9	7	17	16	14
	80	19	19	16	8	8	8	18	16	16
	60	17	17	15	9	8	8	20	15	17
	100W	20	19	17	9	8	9	19	16	17
40 T	100	23	21	20	11	10	10	23	20	21
	80	24	23	20	10	10	10	24	20	22
	80	20	18	18	12	11	12	21	19	19
	60	21	19	20	10	9	10	20	17	19
	130W	23	22	21	9	9	10	20	19	18

(d) SECTION 389, 9.5–6 DESIGN

Axle Load (kips)	Tire Press. (psi)	Strain (10^{-6} in./in.) Compressive Creep	Compressive 15 Mph	Compressive 30 Mph	Tensile Creep	Tensile 15 Mph	Tensile 30 Mph	Deflection (10^{-3} in.) Creep	Deflection 15 Mph	Deflection 30 Mph
18 S	100	20	19	25	11	10	9	26	19	17
	80	21	19	21	11	10	9	26	18	16
	80	24	22	24	14	12	9	28	22	17
	60	22	19	23	11	11	9	28	20	18
	115W	21	24	23	11	10	9	29	21	17
22.4 S	100	23	22	28	12	12	10	30	22	18
	80	28	24	26	13	12	10	32	23	21
	60	31	24	31	14	11	8	32	23	20
	125W	27	23	27	14	12	8	32	23	18
32 T	100	22	21	23	11	11	9	34	27	23
	80	18	18	24	12	12	11	34	25	23
	60	22	18	18	12	12	10	35	27	22
	100W	19	18	23	11	11	10	33	26	23
40 T	100	23	24	29	14	14	12	41	31	27
	80	25	25	28	13	14	13	42	31	28
	80	23	23	23	15	18	13	41	32	27
	60	25	24	27	15	15	14	39	32	28
	130W	24	24	26	20	14	10	40	30	27

[1] Wire cord tire design.

TABLE 32

STRAIN AND DEFLECTION VALUES—SECTION MEANS
RIGID SECTIONS

Tire Press. (psi)	Strain[1] and Deflection[2] Values at Axle Load Indicated:														
	18 S			22.4 S			32 T			40 T			Average		
	Comp.	Tens.	Defl.	Comp.	Tens.	Defl.	Comp.	Tens.	Defl.	Comp.	Tens.	Defl.	Comp.	Tens.	Defl.
(a) SECTION 359, 12.5R–3 DESIGN															
100	15	5	11	18	6	14	15	7	16	20	9	21	17.0	6.8	15.5
80	15	6	12	19	6	14	15	7	17	18	10	21	16.8	7.2	16.0
60	17	5	11	19	7	14	14	7	17	18	9	20	17.0	7.0	15.5
Wire[3]	15	6	12	18	6	14	16	7	17	19	8	21	17.0	6.7	16.0
Avg.	15.5	5.5	11.5	18.5	6.2	14.0	15.0	7.0	16.7	18.7	9.0	20.7			
(b) SECTION 381, 9.5R–3 DESIGN															
100	20	7	13	22	7	13	18	8	16	21	10	21	20.2	8.0	15.7
80	21	7	13	25	7	14	18	8	17	20	11	21	21.0	8.2	16.2
60	21	7	13	23	7	14	16	8	17	20	10	19	20.0	8.0	15.7
Wire[3]	20	6	12	22	7	15	19	9	17	22	9	19	20.7	7.7	15.7
Avg.	20.5	6.7	12.7	23.0	7.0	14.0	17.8	8.2	16.7	20.7	10.0	20.0			
(c) SECTION 367, 9.5–6 DESIGN															
100	24	11	24	27	14	27	19	14	30	25	18	38	23.7	14.2	29.7
80	23	13	25	30	13	27	20	14	31	24	18	36	24.2	14.5	29.7
60	24	13	25	26	14	26	17	14	31	23	16	35	22.5	14.2	29.2
Wire[3]	23	11	23	27	14	27	20	14	32	25	16	35	23.7	13.7	29.2
Avg.	23.5	12.0	24.2	27.5	13.7	26.7	19.0	14.0	31.0	24.2	17.0	36.0			
(d) SECTION 389, 9.5–6 DESIGN															
100	21	10	21	24	11	23	22	10	28	25	13	33	23.0	11.0	26.2
80	22	11	21	26	12	25	20	12	27	24	14	34	23.0	12.2	26.7
60	21	10	22	29	11	25	19	11	28	25	15	33	23.5	11.7	27.0
Wire[3]	23	10	22	26	11	24	20	11	27	25	15	32	23.5	11.7	26.2
Avg.	21.7	10.2	21.5	26.2	11.2	24.2	20.2	11.0	27.5	24.7	14.2	33.0			

[1] In 10^{-6} in. per in.
[2] In 10^{-3} in.
[3] Wire cord tires at pressures 115, 125, 100 and 130 psi (see Table 31).

3.5 RIGID PAVEMENT STUDY

The tests on the rigid pavement sections, given in Table 27, were conducted in the same manner as those on the flexible sections. The schematic drawing (Fig. 25) shows a typical installation of strain gages and deflectometers in one of the rigid sections.

The previously mentioned pilot studies included investigations on rigid sections, and the findings were generally similar to those reported for the flexible sections. Changes in tire inflation pressure from 40 to 70 psi in conventional tires were accompanied by little or no differences in dynamic strain or deflection; but a change in tire pressure from 23 to 70 psi, accompanied by a tire design change from LPLS to conventional, caused significant differences in these measurements.

Table 31 lists the maximum strain (tensile and compressive) and deflection values for the four rigid pavement sections included in the study. Each of the 18 vehicles was operated at three levels of speed at the prescribed transverse placement. Normal relationships of axle load and vehicle speed with tensile or compressive strain and deflection existed. However, the

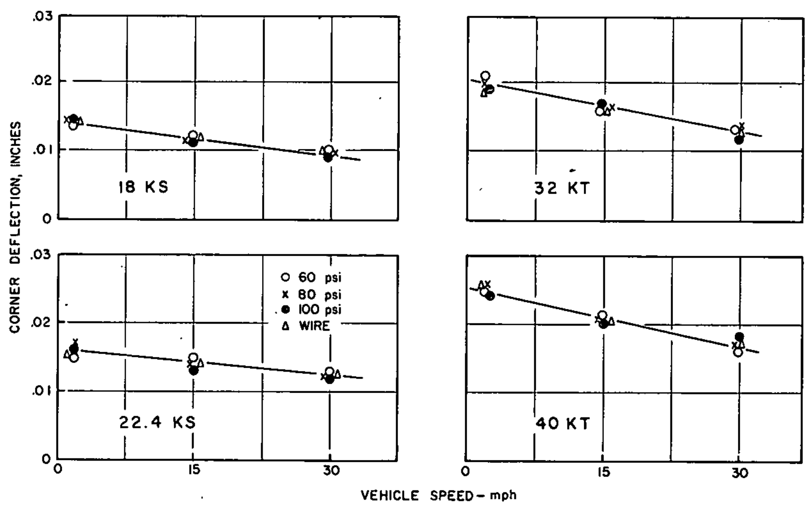


Figure 29. Effect of vehicle speed on corner deflection, Section 359 (12.5 R-3 design).

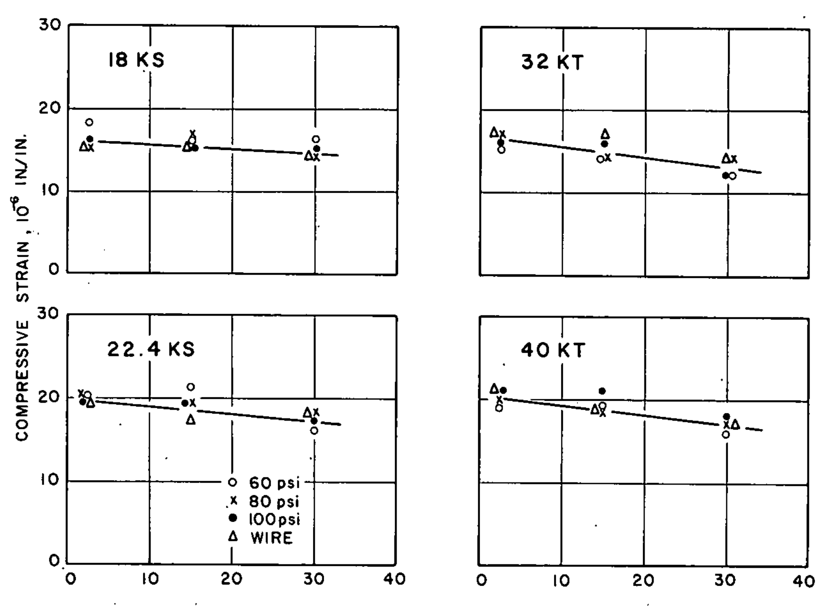


Figure 30. Relationship between vehicle speed and compressive strain, Section 359 (12.5 R-3 design).

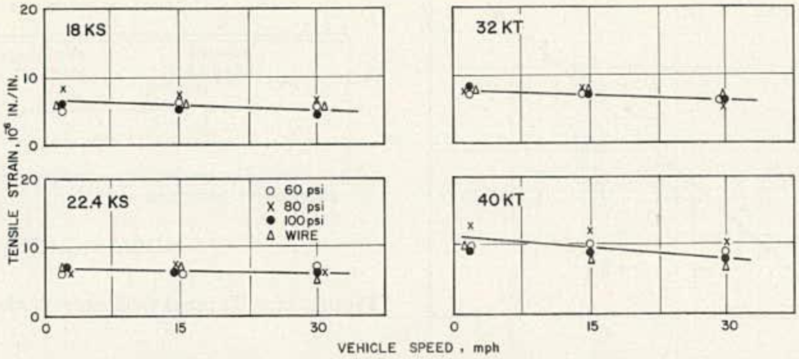


Figure 31. Relationship between vehicle speed and tensile strain, Section 359 (12.5 R-3 design).

data failed to indicate any significant trend in edge strains that could be associated with change in tire pressure or design. Table 32 gives the mean values of strain and deflection for each of the vehicles and tire pressures.

Figure 29 is a typical vehicle speed-deflection relationship for a rigid section under each of the four axle loads. Similar relationships between axle load and both compressive and tensile strain are shown in Figures 30 and 31. From these figures or from the tabular data there did not appear to be a consistent difference attributable to changes in tire pressure or design. These strains and deflections were measured at the pavement edge and/or slab corner. Strains or deflections in other parts of the pavement may or may not have shown tire pressure or tire design effect.

3.6 DYNAMIC LOAD STUDY

Instrumentation was developed at the Road Test by which the dynamic load applied to the pavement structure by a moving tire could be determined. Such measurements offered further means of comparing the effect of the various tire pressures and designs, the equipment was modified and adapted to each of the 16 regular vehicles listed in the tire pressure-tire design group (Figure 32). A comprehensive discussion of the tire pavement interaction based on studies with this equipment is found in Chapter 4 of Road Test Report. Detailed description of the instrumentation used in these studies may be obtained from the Highway Research Board.

Calibration of the dynamic load measuring equipment was required for each vehicle for which it was to be used. Calibration was performed on an electronic scale. One axle of the vehicle equipped with differential tire pressure measuring equipment was placed on the scale and the dead load balanced out in the recording device. A dynamic load was applied, by means of rotating eccentric weights on the vehicle bed, through the tires to the scale platform. An analog record of the change in load was made.

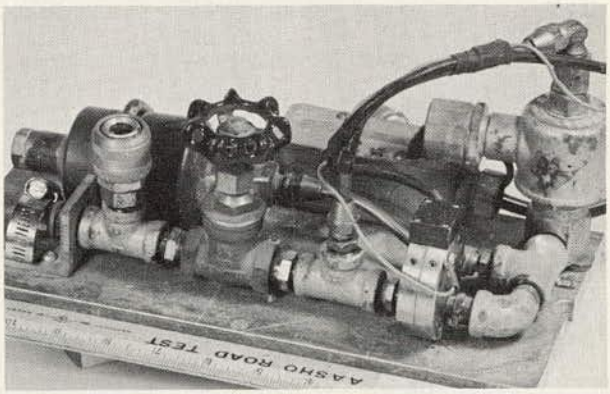


Figure 32. Typical installation of dynamic load—tire pressure equipment; vehicle on electronic scale during calibration.

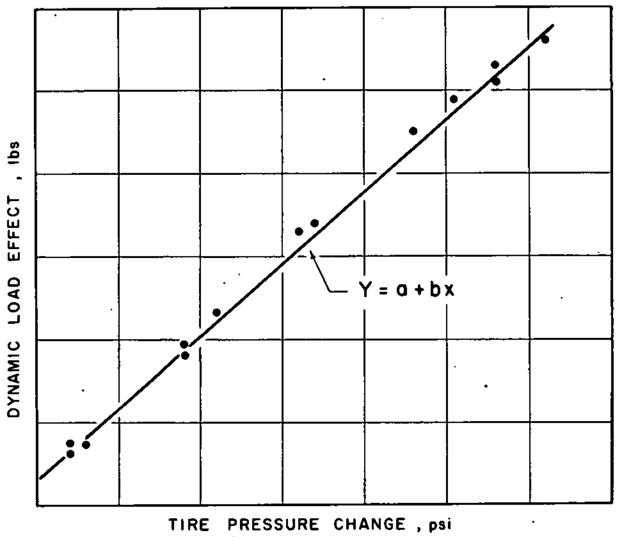


Figure 33. Typical dynamic load—tire pressure calibration curve.

Simultaneously a record was made of the change in tire pressures. From these records it was possible to relate the change in load to the change in tire pressure. For the various tires tested this relationship was linear over the range of load applied. Figure 33 is a typical calibration curve.

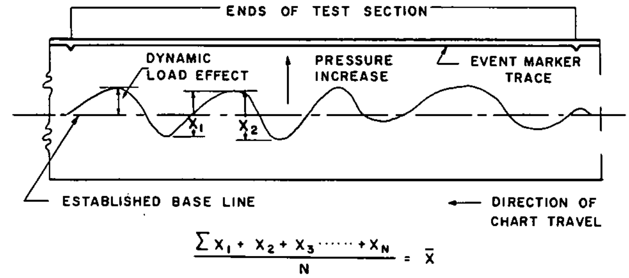


Figure 34. Typical field record, change in tire pressure.

During the field runs over the test sections with the various vehicles, a continuous record was made of the change in tire pressure. The mean amplitude, from peak to valley, of the tire pressure change records was computed for each run over a test section. The average amplitude for the load change for the run was then determined from the corresponding calibration curve. One-half of the amplitude of the average dynamic load change could then be added to or subtracted from the static load in order to determine the dynamic load range to which the particular pavement had been subjected during that run. It is this half amplitude of load change that is referred to in this report as the dynamic load effect. The foregoing procedure is shown in Figure 34. The following example

TABLE 33

MEAN DYNAMIC LOAD EFFECT

Axle Load (kips)	Tire Press. (psi)	Tire Size[1] (in.)	Tire Contact Area per Vehicle (sq. in.)	Mean Dynamic Load Effect (lb)			
				Rough Pavement		Smooth Pavement	
				10 Mph	30 Mph	10 Mph	30 Mph
18 S	60	12.00x20N	335.2	—	3,020	—	3,120
	80	10.00x20N	285.2	3,580	4,150	2,960	3,570
	100	9.00x20N	254.0	3,420	3,480	2,270	3,670
	115W	8.25x20W	220.0	2,480	3,960	1,500	3,730
22.4 S	60	12.00x20N	368.0	4,660	5,400	1,560	3,100
	80	11.00x20N	256.8	4,880	7,140	2,260	4,000
	100	10.00x20N	284.8	880	1,370	610	660
	125W	9.00x20W	244.4	3,540	7,400	2,400	4,580
32 T	60	11.00x20N	462.4	2,090	2,230	2,280	2,230
	80	9.00x20N	469.6	1,710	2,320	1,140	1,960
	100	8.25x20N	370.4	2,790	3,460	1,620	3,100
	100W	8.25x20W	381.6	2,380	2,950	1,240	2,970
40 T	60	12.00x20N	630.4	2,240	6,480	830	2,130
	80	11.00x20N	552.0	2,480	4,920	1,250	2,810
	100	9.00x20N	479.2	2,510	5,640	1,790	3,310
	130W	8.25x20W	447.2	2,000	1,930	1,450	2,680

[1] N = nylon; W = wire.

is given to illustrate the terminology used. Assuming that a particular vehicle operating over a particular section had a static axle load of 18 kips and that the tire pressure change from maximum to minimum averaged over all of the cycles recorded 0.25 lb per sq in., from the calibration curve for this vehicle a tire pressure change of 0.25 lb per sq in. corresponded to load change of 4,000 lb. Presumably one-half of this load change applied above the static axle load and one-half below; therefore, the dynamic effect was 2,000 lb. This means that during the test run the pavement was subjected to axle loads varying from 16,000 lb to 20,000 lb. In this case, the percentage dynamic effect is equal to 2,000 divided by 18,000, or about 11 percent.

The vehicles equipped to record tires pressure changes were operated over two sections of pavement at speeds of 10 and 30 mph. The two sections were generally classified as rough and smooth, having serviceability indexes of 2.71 and 4.66, respectively.

Table 33 gives the mean dynamic effect for the vehicles tested at both speed levels on both of the test sections. The values for the tandem axles are the means of both loaded axles.

In general, the data from the study appear rational in that the mean dynamic load effect for the rough pavement is greater than for the smooth pavement, and an increase in speed from 10 to 30 mph is accompanied by an increase in dynamic load effect. Variations from the rule can be attributed to the difficulty in maintaining exact speeds and transverse placements in subsequent vehicle passages.

The objective of this study was to investigate the effect of changes in tire pressure and design on the dynamic load effect.

The mean percentage of increase in dynamic load effect over the static axle load at each tire inflation pressure was computed for every load as given in Table 34. No consistent trend related to inflation pressure or tire design was found.

The magnitude of the dynamic load effect was of interest. At speeds below highway speeds, the actual load on the pavement changed as much as 30 percent above and below the static load at a rate of 2 to 4 cycles per second.

3.7 BRIDGE STUDY

Midspan strain and deflection were selected as variables to be measured to determine the dynamic effect, if any, of changes in tire pressure and designs on the bridge structures.

The 16 vehicles (not including two replicates) in this study were operated over the remaining test structures in Loop 6, (Table 27) at three levels of speed at a transverse location symmetrical with respect to the three bridge beams. Figure 35 is a schematic drawing of the instrument installations on the bridge beams.

The responses of gages on all three beams and on all three deflectometers were recorded

TABLE 34

PERCENT INCREASE IN DYNAMIC AXLE LOAD

Axle Load (kips)	Tire Press. (psi)	Tire Size[1] (in.)	Axle Load Increase (%)				
			Rough Pavement		Smooth Pavement		Mean
			10 Mph	30 Mph	10 Mph	30 Mph	
18 S	60	12.00x20N	—	16.8	—	17.3	17.0
	80	10.00x20N	19.9	23.1	16.5	19.8	19.8
	100	9.00x20N	19.0	19.3	12.6	20.4	17.8
	115W	8.25x20W	13.8	22.0	8.3	20.2	16.1
22.4 S	60	12.00x20N	20.9	24.1	7.0	14.5	16.6
	80	11.00x20N	21.9	31.9	10.1	17.9	20.4
	100	10.00x20N	3.9	—	2.7	—	3.3
	125W	9.00x20W	15.9	33.0	10.8	20.5	20.0
32 T	60	11.00x20N	13.1	13.9	14.2	13.9	13.8
	80	9.00x20N	10.7	14.5	7.1	12.3	11.2
	100	8.25x20N	17.5	21.6	10.1	19.4	17.2
	100W	8.25x20W	14.9	18.4	7.7	18.5	14.9
40 T	60	12.00x20N	11.2	32.4	4.2	10.7	14.6
	80	11.00x20N	12.4	24.6	6.3	14.1	14.4
	100	9.00x20N	12.6	28.2	9.0	16.5	16.6
	130W	8.25x20W	10.0	—	7.3	13.4	10.2

[1] N = nylon; W = wire.

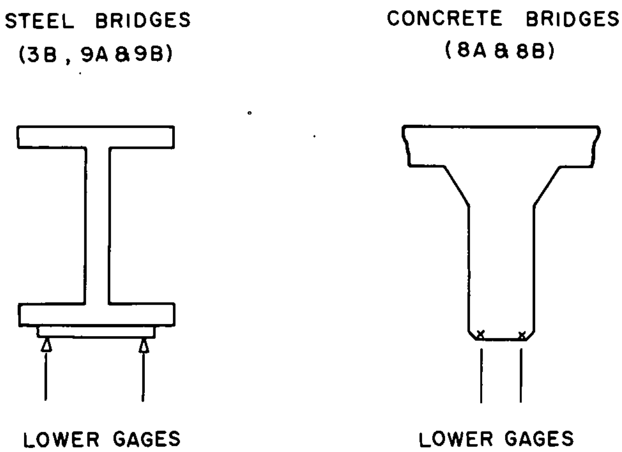


Figure 35. Location of midspan gages.

simultaneously and were averaged to give the mean strain or deflection caused by each vehicle.

Throughout this report the dynamic effects produced in the bridges under the influence of moving loads are expressed in terms of the corresponding effects produced at creep speed. This ratio is defined as either the strain or deflection amplification factor.

Any apparent effect of the tire pressure or design on the responses measured is subject to question because the characteristics of the vehicles were not directly comparable. For example, only the trailer axle or axles were loaded and equipped with the specified tire design at the given pressure and the variability in the spring constants for the vehicles was not determined. Since the maximum moment at midspan is related to the gross vehicle load and the position of the axles with respect to midspan, the dynamic measurements of strain and deflection, or any variation in these, are related to the same factors.

The deflection and strain amplification factors were computed for the 16 vehicles and the 5 structures in the study. Table 35 gives the amplification factors for Bridge 3B. These data are presented as examples of the amplification factors for the other four structures. The amplification factors for Bridge 3B and for the other structures show little variation with respect to change in tire pressure or tire design. Table 36 gives the mean amplification factors for each of four tire pressures for the five structures. The wire cord tires have been grouped in this table. The pressure range of these tires is 100 to 130 psi.

Columns 12 and 13 present the mean amplification factors for all units at a given tire pressure for all structures at both levels of speed. A study of these means shows there was little variation in amplification factors with change in tire pressure. Furthermore, no consistent trend with change in tire pressure existed for any of the axle loads. The effect of vehicle speed on the amplification factors is evident when these two columns are compared.

TABLE 35

MEAN AMPLIFICATION FACTORS FOR BRIDGE 3B, LOOP 6

Axle Load (kips)	Tire Press. (psi)	Amplification Factor[1]			
		Strain		Deflection	
		15 Mph	30 Mph	15 Mph	30 Mph
18 S	100	1.097	1.219	1.156	1.218
18 S	80	1.142	1.190	1.218	1.218
18 S	60	1.170	1.195	1.250	1.281
18 S	115W	1.120	1.195	1.225	1.258
22.4 S	100	1.112	1.224	1.162	1.297
22.4 S	80	1.145	1.288	1.194	1.416
22.4 S	60	1.148	1.212	1.222	1.305
22.4 S	125W	1.142	1.163	1.162	1.297
32 T	100	1.047	1.190	1.063	1.170
32 T	80	1.063	1.015	1.085	1.191
32 T	60	1.083	1.166	1.111	1.177
32 T	100W	1.031	1.174	1.130	1.217
40 T	100	1.037	1.139	1.087	1.175
40 T	80	0.963	1.048	1.069	1.206
40 T	60	1.050	1.139	1.086	1.224
40 T	130W	1.051	1.155	1.105	1.210

[1] Ratio of moving load dynamic effects to creep speed dynamic effects.

The same data are given in Table 37 summarized for comparison of the effect of class of vehicle on the strain and deflection amplification factors. The means for all structures (Columns 12 and 13) show that the amplification factors for the tandem axle vehicles are lower than those for the single axle vehicles. This finding agrees with that given in AASHO Road Test Report 4. The individual means for Bridge 8A show an unexplained reversal of this relationship.

3.8 NEEDED RESEARCH

Although some conclusive trends were shown by this study, it was not clearly established how changes in tire pressure and tire design affected the pavement structure.

Further research should include a greater range of tire pressures and designs than were available for this study. Also, the selection of the pavement sections should include designs below the thickness of those that were available on the Road Test at the time of this study.

Some of the data presented might be of further value if different analyses were carried out. In particular, a different means of summarizing the dynamic load records might establish more definite trends.

In addition, further research to detect these effects should include performance studies patterned after the experiment design of the main Road Test.

TABLE 36

MEAN AMPLIFICATION FACTORS FOR ALL BRIDGES, LOOP 6, BY TIRE PRESSURE

Tire Press. (psi)	Bridge 3B		Bridge 8A		Bridge 8B		Bridge 9A		Bridge 9B		All Bridges	
	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*	*(13)*
						(a) STRAIN AMPLIFICATION FACTOR						
100	1.073	1.193	1.073	1.044	1.045	1.064	1.052	1.080	1.030	1.104	1.054	1.097
80	1.078	1.135	1.045	1.024	1.035	1.083	1.026	1.038	1.038	1.029	1.044	1.061
60	1.112	1.165	1.046	1.025	1.033	1.134	1.003	1.059	1.031	1.066	1.045	1.089
Wire [1]	1.086	1.162	1.044	1.004	1.069	1.146	1.027	1.071	1.039	1.086	1.053	1.093
						(b) DEFLECTION AMPLIFICATION FACTOR						
100	1.117	1.215	1.090	1.116	1.077	1.060	1.109	1.120	1.067	1.164	1.092	1.135
80	1.141	1.257	1.092	1.091	1.107	1.097	1.056	1.040	1.076	1.159	1.094	1.128
60	1.167	1.246	1.121	1.100	1.116	1.139	1.053	1.100	1.060	1.147	1.103	1.146
Wire [1]	1.155	1.245	1.117	1.101	1.137	1.152	1.112	1.125	1.062	1.166	1.116	1.157

[1] Tire pressures: 18KS, 115 psi; 22.4KS, 125 psi; 32KT, 100 psi; 40KT 130 psi.

TABLE 37

MEAN AMPLIFICATION FACTORS FOR ALL BRIDGES, LOOP 6, BY VEHICLE CLASS

Vehicle Class	Bridge 3B		Bridge 8A		Bridge 8B		Bridge 9A		Bridge 9B		All Bridges	
	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph	15 Mph	30 Mph
(1)	*(2)*	*(3)*	*(4)*	*(5)*	*(6)*	*(7)*	*(8)*	*(9)*	*(10)*	*(11)*	*(12)*	*(13)*
						(a) STRAIN AMPLIFICATION FACTOR						
Single [1] axle	1.135	1.210	1.049	1.016	1.081	1.108	1.037	1.086	1.066	1.148	1.073	1.113
Tandem [2] Axle	1.054	1.128	1.056	1.033	1.010	1.106	1.018	1.032	1.015	1.038	1.030	1.067
						(b) DEFLECTION AMPLIFICATION FACTOR						
Single [1] Axle	1.198	1.286	1.107	1.101	1.162	1.135	1.087	1.107	1.067	1.205	1.124	1.166
Tandem [2] Axle	1.093	1.196	1.092	1.104	1.057	1.091	1.079	1.071	1.067	1.075	1.077	1.107

[1] 18 and 22.4 kips. [2] 32 and 40 kips

Chapter 4

Commercial Construction Equipment

4.1 SUMMARY

The objective of the study was to determine the dynamic effect on bridges and pavements of commercial construction equipment and, insofar as possible, to relate the dynamic effects of these vehicles to those observed for conventional dual-tire truck units.

One medium and one small 2-axle tractor-scraper units were operated over pavement sections on which dynamic measurements were made of strain, deflection, transmitted embankment pressure, and dynamic load effect. Maximum strain and deflection measurements were taken on the test structure.

Flexible Pavement Study

The relationships between deflection and wheel load for the two scraper units agreed with the relationships for conventional truck units. The rate of increase of deflection with wheel load for the small scraper was essentially equal to that for the conventional units; whereas, the rate for the medium scraper was considerably lower than the rate for the conventional units.

With the instrumentation available, the study of the effect of a change in tire pressure on the pavement deflection failed to indicate any trends at any wheel load or inflation pressure tested.

For both units deflection decreased as vehicle speed increased. However, the effect of vehicle speed was more pronounced for the small than for the medium scraper. In addition, the speed effect was greater at the higher wheel loads.

As vehicle speed increased there was a decrease in pressure transmitted to the embankment by both scrapers at all levels of wheel load and tire inflation pressure tested. However, changes in inflation pressure did not noticeably affect the transmitted pressure.

Rigid Pavement Study

For both scrapers, compressive edge strains increased with wheel load, but were affected very little by vehicle speed or tire inflation pressure. Tensile strains (measured at the pavement edge) were not noticeably affected by wheel load, vehicle speed, or inflation pressure.

Corner deflection measurements increased with an increase in wheel load for both scrapers at a lower rate than for the conventional units, but the effect of inflation pressure was neither uniform nor large for either scraper. An increase in vehicle speed caused a decrease in corner deflection at a uniform rate for both scrapers at both inflation pressures.

Dynamic Load Study

The percent of increase in dynamic axle load over static load was comparable to that found for the conventional truck units. Of the two sections of pavement tested, the dynamic load effect was appreciably greater for the pavement with lower serviceability, and also increased appreciably with an increase in vehicle speed or an increase in inflation pressure.

Bridge Study

Although the trend was not consistent for all bridges tested, the mean strain and deflection amplification factors for both scraper units were lower than those for the single axle vehicles but greater than those for the tandem axle vehicles.

The mean strain amplification factors for the small scraper unit were slightly higher than for the medium unit. However, the relationship was not consistent for the deflection amplification factors.

4.2 SCOPE

Units of commercial construction equipment were made available by the manufacturer at the request of the Department of Defense for this portion of the study.

One medium and one small 2-axle tractor scraper units with struck capacity of 21 and 14 cu yd, respectively, were selected. Table 38 shows characteristics of these units at various load and tire inflation pressure levels and the characteristics of the conventional tractor-semitrailer truck units used for comparative purposes.

To evaluate the effect on pavements and bridges of the different units at different levels of speed, load and tire pressure, an investigation of strain, deflection, embankment pressure and dynamic load change was conducted on pavement sections in Loops 4 and 6 and on certain test bridges in Loop 6.

4.3 DESCRIPTION OF MEASUREMENTS

Table 39 gives the test sections and structures selected for the study and the type of

TABLE 38

VEHICLE CHARACTERISTICS

Vehicle	Tire Size[1] (in.)	Tire Press. (psi)	Gross Load (kips)	Axle Load (kips)					Axle Spacing (in.)				Center to Center of Duals or Tires (in.)
				1	2	3	4	5	1–2	2–3	3–4	4–5	
Conv.	7.50x20N	80	28.1	3.7	12.4	12.0			142	252			69
Conv.	10.00x20N	80	40.3	5.6	16.8	17.9			143	246.5			72
Conv.	9.00x20N	80	74.5	9.5	16.4	16.1	15.4	17.1	129	48	241	50	72
Conv.	11.00x20N	80	90.3	9.0	20.4	20.1	20.7	20.1	144	50	241	50	72
Small scraper	26.5 x25	45	87.9	47.0	40.9				269				78
		45	67.6	39.6	28.0								
		30	67.6	39.6	28.0								
		45	48.3	32.9	15.4								
		30	48.3	32.9	15.4								
Medium scraper	29.5x35	45	122.5	67.5	55.0				292				87
		45	87.5	52.5	35.0								
		30	87.5	52.5	35.0								
		45	67.0	45.6	21.4								
		30	67.0	45.6	21.4								

[1] N = nylon cord.

TABLE 39

INSTRUMENTED SECTIONS AND STRUCTURES

Loop	Section	Tangent	Thickness (in.)			Measurement
			Surface	Base	Subbase	
		(a) SECTIONS				
6	265	Flexible	5	9	16	Deflection
	271	Flexible	6	9	8	Deflection
	301	Flexible	6	6	16	Deflection
	333	Flexible	6	9	16	Deflection
4	581	Flexible	5	6	12	Embankment pressures
6	367	Rigid	9.5		6	Strain and deflection
	381	Rigid	9.5		3	Strain and deflection
	389	Rigid	9.5		6	Strain and deflection
	397	Rigid	11.0		6	Strain and deflection
		(b) BRIDGES				
6	3B	Steel-composite, 27 ksi, cover plate 18WF60				Strain and deflection
	8A	Reinforced conc. monolithic, 30 ksi, 3 No. 11, 2 No. 1, 1 No. 8				Strain and deflection
	8B	Reinforced conc. monolithic, 30 ksi, 3 No. 11, 2 No. 9, 1 No. 8				Strain and deflection
	9A	Steel-noncomposite, 27 ksi, cover plates 18WF96				Strain and deflection
	9B	Steel-noncomposite, 27 ksi, cover plates 18WF96				Strain and deflection

measurements taken in each section. Figure 25 is a schematic drawing of typical instrument installations in both the rigid and flexible pavements.

The vehicles were operated over the instrumented sections at creep speeds and at 15 mph (considered to be safe speeds for the construction equipment at all load levels).

Temperature variations in the pavement structure, which might have affected the deflection and strain values, were minimized for each test section by conducting the study under closely controlled conditions. However, section-to-section analysis of the thickness-deflection relationships was not possible because there were wide temperature differences from section to section.

The data and findings are reported for the flexible pavements, rigid pavements, dynamic load study and for the bridge study.

4.4 FLEXIBLE PAVEMENT STUDY

Relationships were developed between vehicle speed and deflection, axle load (wheel load) and deflection; and studies were made of relative dynamic load and pressure transmitted to the embankment soil for each of the construction units. A comparison was made of these effects with those for the conventional vehicle units.

Table 40 is a summary of the field deflection readings from the four flexible sections of Loop 6. Because of the great tread width of the scraper units, only the inner wheel path total deflection values are listed. The data for pavement sections 265 and 333 are complete for the entire range of inflation pressures and axle loads. These sections were selected because their serviceabilities were high and because they were conveniently located.

TABLE 40

TOTAL DEFLECTION [1], FLEXIBLE SECTIONS

Vehicle	Axle Load (kips)	Tire Press. (psi)	Total Deflection (10^{-3} in.) Sect. 265 Creep	Sect. 265 15 Mph	Sect. 333 Creep	Sect. 333 15 Mph	Sect. 271 Creep	Sect. 271 15 Mph	Sect. 301 Creep	Sect. 301 15 Mph
Conv.	12	80	11	8	18	14	18	13	15	10
	16 [2]	80	15	13	23	17	28	20	20	17
	18	80	17	13	27	19	28	26	21	18
	20 [3]	80	21	20	30	21	40	30	27	22
Small scraper	15.4	45	20	19	24	20				
	28.0	45	33	25	35	30				
	32.9	45	33	31	39	31				
	39.6	45	39	34	44	33				
	40.9	45	39	33	45	35	73	57	43	40
	47.0	45	45	36	51	38	78	62	48	45
Medium scraper	21.4	45	19	18						
	35.0	45	36	23						
	45.6	45	32	27						
	52.5	45	43	26						
	55.0	45	43	33	39	41	77	65	44	40
	67.5	45	41	33	34	41	78	63	42	35
Small scraper	15.4	30	18	16	23	16				
	28.0	30	31	28	41	30				
	32.9	30	34	29	39	31				
	39.6	30	36	34	44	35				
Medium scraper	21.4	30	22	15						
	35.0	30	36	29						
	45.6	30	35	26						
	52.5	30	36	29						

[1] IWP values only; mean of a minimum of four field readings.
[2] Tandem axle load of 32 kips.
[3] Tandem axle load of 40 kips.

Both axles of the scraper units were equipped with identical tires and were considered to act equally in all respects. Thus, changing the gross load on the vehicle was a convenient means of obtaining two additional axle loads for observation. A maximum of six deflection value means at different wheel loads were used to develop the relationships shown in Figures 36 and 37 between wheel load and deflection. The curves shown were developed for both units assuming a power function relationship between wheel load and deflection (see Report 5).

The data for both inflation pressures for each unit were combined on a log-log plot, and the straight lines were drawn by eye. The resultant curves were then displayed on the rectilinear plots.

For the small scraper, the curve of deflection with wheel load was nearly parallel, but higher than the curve for the conventional truck units at both speed levels. However, a similar comparison for the medium unit shows an appreciably greater rate of change of deflection with changing loads for the conventional units at both speed levels.

The scatter of the plotted points (each point is the mean of at least four field readings) for the medium unit can be attributed almost entirely to the difficulty of maintaining identical transverse placement of the two axles of the unit.

The curves in Figures 36 and 37 have been redrawn in Figure 38 to show the relative effects of vehicle speed for each unit.

Pavement deflection may be considered to decrease exponentially as speed increases. That is, a greater speed effect is indicated at the higher wheel loads than at smaller wheel loads for both units (see Report 5).

The effect of vehicle speed on deflection is greater for the medium unit than for the small scraper at all levels of wheel load. A study of the data offers no explanation for this phenomenon.

The data shown on the previous plots are from test section 265 in Loop 6. Similar relationships and observations can be shown for the small scraper unit from the data from test section 333.

Studies of the maximum pressure transmitted to the embankment soil and influence area studies were conducted for the construction equipment.

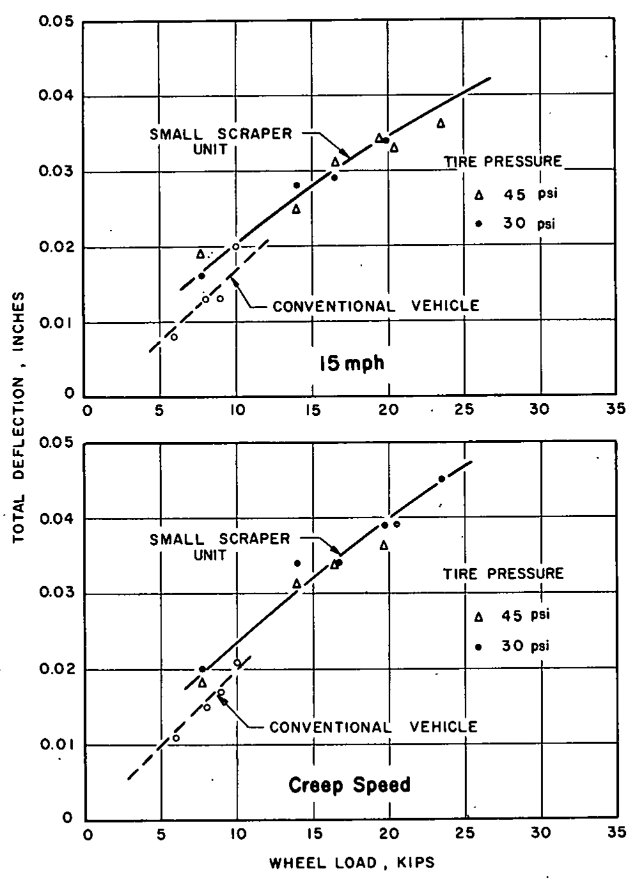


Figure 36. Relationship between wheel load and deflection, Section 265 (5-9-16 design).

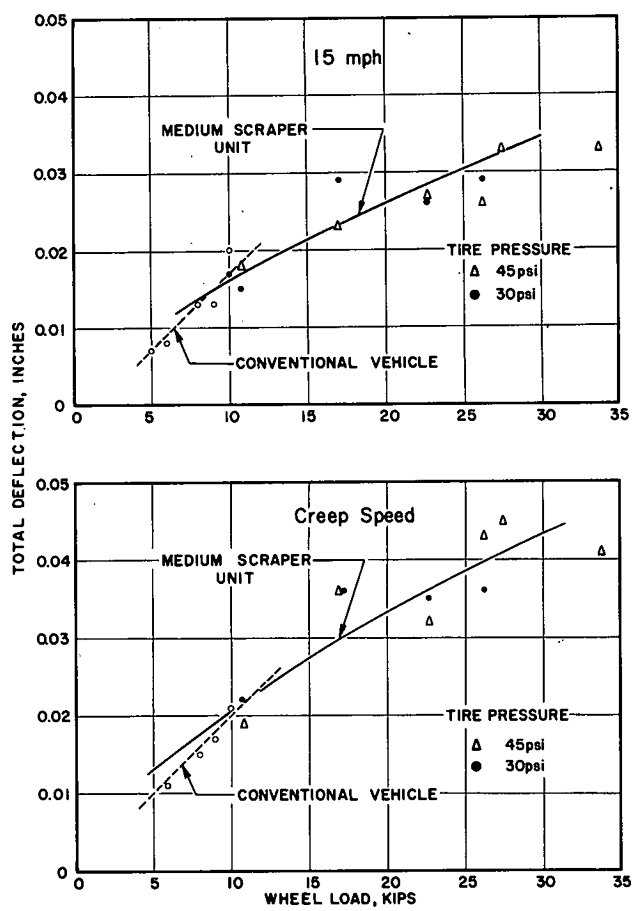


Figure 37. Relationship between wheel load and deflection, Section 265 (5-9-16 design).

Table 41 gives the maximum values of transmitted pressure for every axle load, speed and vehicle included in the study, as well as the distance at which the measured pressure was zero as shown by the analog records of pressure.

The relationship of vehicle speed to transmitted pressure is shown in Figures 39 and 40 for both the small and medium scraper units. This relationship cannot be developed because only two levels of speed were included in the study, but the offset in the lines for the different speeds at each of the two inflation pressures indicates that the effect of the vehicle speed on the transmitted pressure is reasonably linear at all levels of wheel load tested.

Figures 41 and 42 are redrawn from Figures 39 and 40 to show the relationships for the transmitted pressure and wheel load at both inflation pressures for both units. There does not appear to be a significant difference between the transmitted embankment pressures developed under the two tire inflation pressures regardless of speed. The wheel load-transmitted pressure relationship for the conventional dual tire units is shown for comparison.

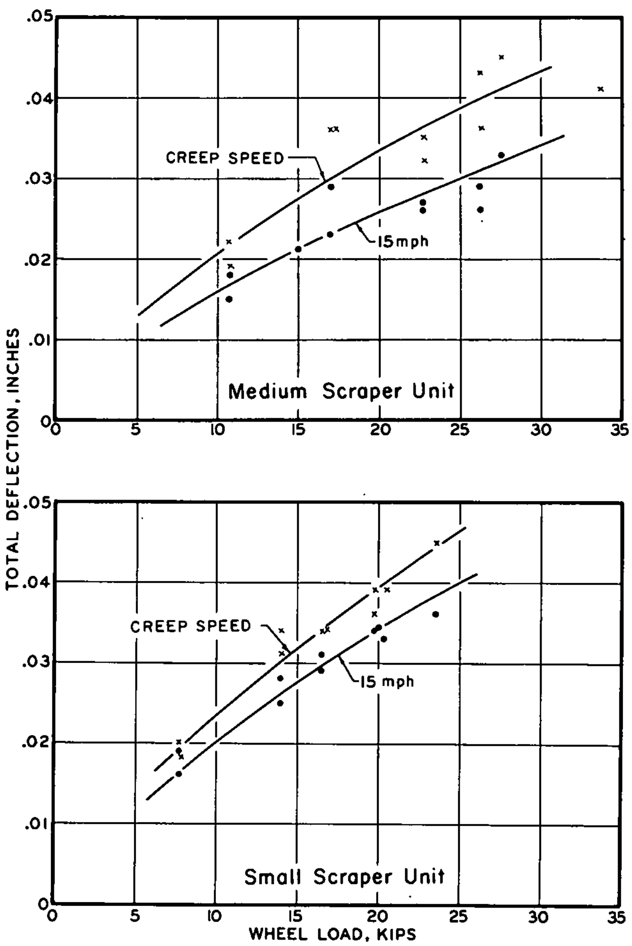


Figure 38. Effect of vehicle speed on deflection, Section 265 (5-9-16 design).

Table 41 shows the longitudinal distance from the loaded axle at which zero pressure at the embankment level was recorded. This distance appeared to vary linearly with wheel load with very little effect of vehicle speed. The same relationship was found for the conventional dual-tire units.

Figure 43 shows influence diagrams for wheel loads of 15.4 and 39.6 kips at two levels of inflation pressure for the small scraper unit. Changes in pressures and distances between the influence lines can be used to substantiate the wheel load effect. There appears to be little or no effect of inflation pressure on the pressure transmitted to the embankment at either of the wheel loads tested. Similar diagrams and effects can be shown for all axle loads and speeds for both units.

4.5 RIGID PAVEMENT STUDY

Table 42 shows deflections and strain data for the construction equipment obtained on test section 397, Loop 6. Other sections were included in the study, but day-to-day temperature variations limited the usefulness of the data for making comparisons with the conventional vehicles.

Figure 44 shows the relationship of wheel load and compressive strain for both scraper units at two levels of vehicle speed. The effect of the tire inflation pressure on the increase of strain with wheel load was very slight for these data. The line representing the strains recorded under the conventional units is shown to permit a comparison. In all instances the slope of the line is about the same as for the construction equipment, but the strains themselves are somewhat greater.

Relationships for the tensile strain and wheel loads could not be determined from the data obtained.

Figure 45 presents the relationships of total deflection (slab corner) with wheel load. A comparison with data obtained from the conventional units showed a lower rate of increase in deflection with wheel load for the construction equipment. The effect of the tire inflation pressure did not appear to be uniform for both units. A divergent effect was noted for the small unit as the load was increased; whereas, the effect was essentially constant throughout the entire wheel load range for the medium unit.

A similar observation can be made for the relationships shown in Figure 46. For the medium unit, uniform effects of speed, tire pressure, and wheel load on deflection are shown.

4.6 DYNAMIC LOAD STUDY

A very limited study of dynamic load applied to the pavement was conducted using the small

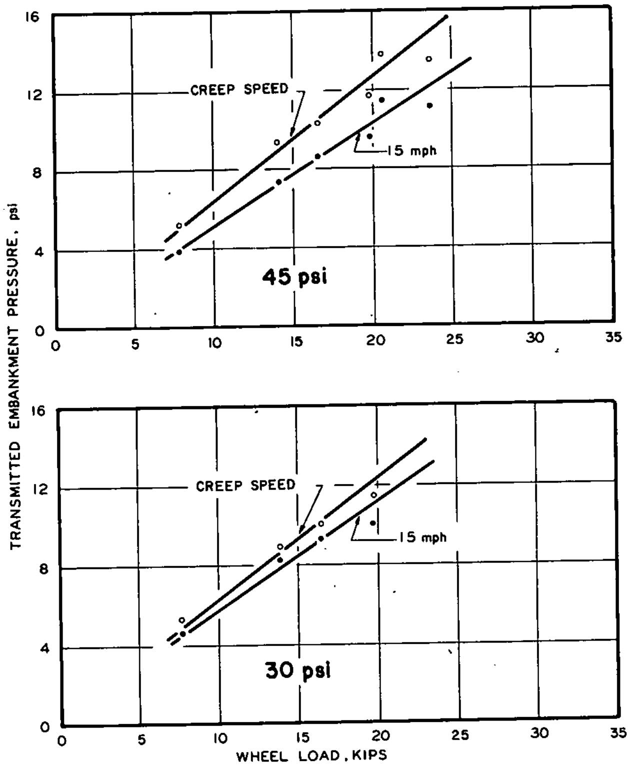


Figure 39. Relationship between wheel load and embankment pressure, Section 581 (5-6-12 design), small scraper unit.

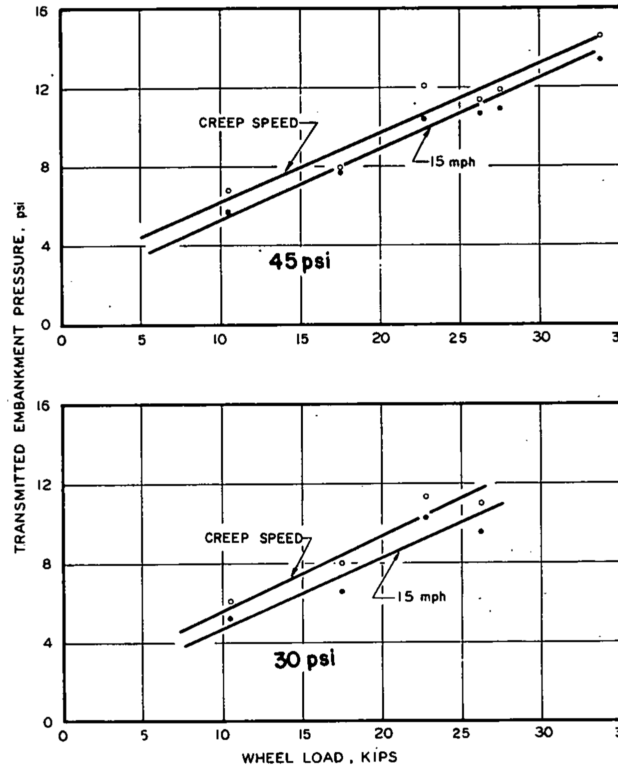


Figure 40. Relationship between wheel load and embankment pressure, Section 581 (5-6-12 design), medium scraper unit.

TABLE 41

TRANSMITTED EMBANKMENT PRESSURES, SECTION 581 (5-6-12 DESIGN)

Vehicle	Axle Load (kips)	Tire Press. (psi)	Zero[1] Pressure Reading (ft)		Max. Press. (psi)		Zero[2] Pressure Reading (ft)	
			Creep	15 Mph	Creep	15 Mph	Creep	15 Mph
Small scraper	15.4	45	4.4	5.2	5.2	3.9	5.2	5.0
		30	4.4	4.8	5.3	4.6	5.2	4.8
	28.0	45	4.8	4.8	9.4	7.3	5.2	5.6
		30	4.8	5.2	8.9	8.2	5.2	5.6
	32.9	45	5.2	5.8	10.3	8.5	5.6	5.8
		30	5.6	4.8	10.0	9.3	6.0	5.6
	39.6	45	4.8	5.2	11.7	9.6	5.2	5.2
		30	5.6	5.6	11.4	10.0	5.6	6.0
	40.9	45	5.9	6.0	13.8	11.4	5.9	6.0
	47.0	45	6.4	6.0	13.5	11.1	6.4	6.0
Medium scraper	21.4	45	5.0	5.9	6.8	5.7	5.0	5.9
		30	5.5	5.2	6.1	5.3	6.0	5.6
	35.0	45	6.5	5.5	7.8	7.8	6.5	6.4
		30	7.0	6.0	8.0	6.6	7.0	6.5
	45.6	45	5.5	5.9	12.1	11.4	5.5	6.4
		30	6.0	5.2	11.4	10.3	6.5	6.0
	52.5	45	6.5	6.4	11.4	10.7	7.0	7.3
		30	7.0	6.0	11.5	9.6	7.0	7.0
	55.0	45	6.5	6.5	11.9	10.9	6.5	7.0
	67.5	45	7.0	7.0	14.6	13.4	7.0	7.0
Conv.	12.0 S	80	4.0	5.0	4.2	3.4	4.6	6.0
	18.0 S	80	4.0	4.6	5.3	4.8	5.0	6.0
	32.0 T	80	4.3	5.3	4.9	3.7	4.8	6.6
	40.0 T	80	5.0	5.0	5.1	4.6	5.2	5.5

[1] Leading distance to 0 pressure.
[2] Trailing distance to 0 pressure.

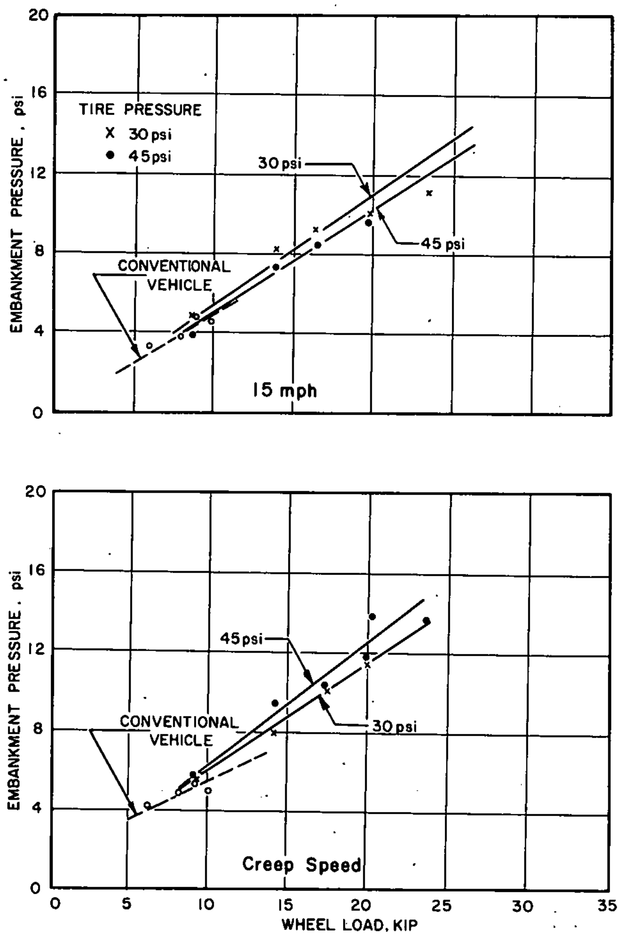


Figure 41. Effect of tire pressure on embankment pressure, small scraper unit, Section 581 (5-6-12 design).

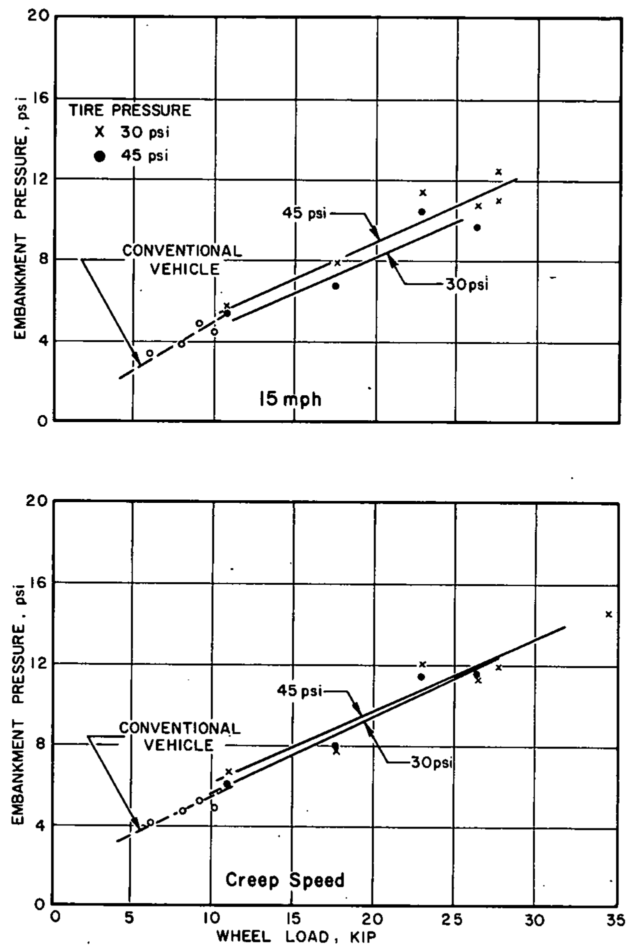


Figure 42. Effect of tire pressure on embankment pressure, medium scraper unit, Section 581 (5-6-12 design).

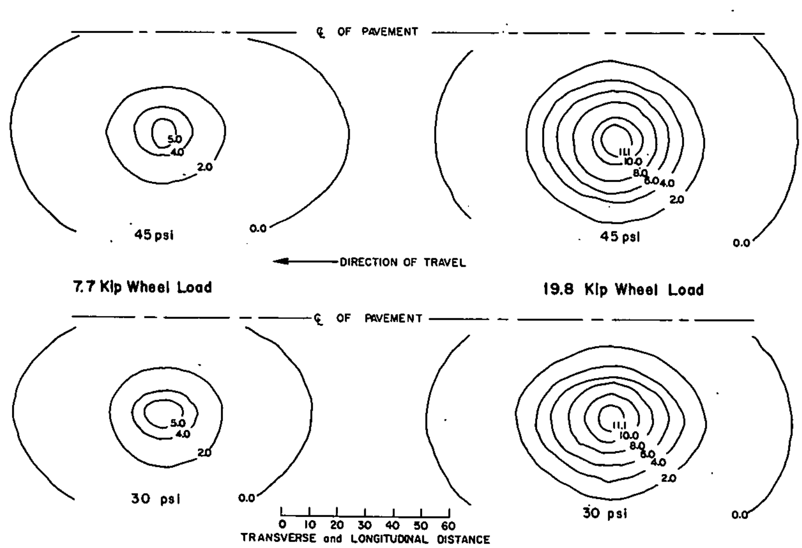


Figure 43. Embankment pressure influence diagrams, small scraper unit, Section 581 (5-6-12 design).

scraper unit. The device described in Chapter 3 was further modified to fit the tires on this unit. Because of the rigid rear axle on the scraper, the instrumentation was applied only to the tractor or drive axle. The axle load selected for study was 39.6 kips.

The scraper unit was operated over two selected sections of pavement (rough and smooth) on Loop 6 at speeds of 10 and 20 mph at both 30- and 45-psi inflation pressure. Table 43 gives dynamic load effect for each of the speeds and inflation pressures.

Very little significance can be attached to dynamic load effect differences between tire inflation pressures and vehicle speeds unless the values for the rough pavement section at 20 mph are considered as extreme. These data are subject to variations due to nonuniform vehicle speed and variable transverse vehicle placement to a greater extent than are the data for the other speed and section conditions. With this thought in mind, an increase in tire inflation pressure appeared to cause a highly significant increase in dynamic load effect.

The serviceabilities of the sections tested were noted in Chapter 3 as 2.71 and 4.66. A definite difference in dynamic load effect related to differences in serviceability was evident from the data. The effect of vehicle speed was similar to that reported for the conventional units in Chapter 3.

Table 44 lists the percent of change of axle load (over the static condition) for the various speed and tire inflation pressure conditions. The percentages shown are similar to those reported for the conventional units in Chapter 3, but no direct comparison can be made because of the large difference in wheel or axle loads.

TABLE 42

STRAIN AND DEFLECTION VALUES, RIGID SECTION 397 (11-6 DESIGN)

Vehicle	Axle Load (kips)	Strain (10^{-6} in./in.) Compressive		Tensile		Deflection (10^{-3} in.)	
		Creep	15 Mph	Creep	15 Mph	Creep	15 Mph
Conv.	12 S	11	11	6	5	10	10
	32 T	17	17	9	6	20	20
	18 S	17	17	6	5	15	14
	40 T	23	20	16	15	32	29
Small scraper, 45 psi	15.4	19	13	10	9	10	9
	28.0						
	32.9	27	25	10	9	20	19
	39.6						
	40.9	40	40	18	9	37	34
	47.0	48	48	16	17	38	35
30 psi	15.4	13	14	11	10	11	11
	28.0	26	25	13	13	25	21
	32.9	31	30	11	10	23	20
	39.6	38	35	13	13	30	27
Medium scraper, 45 psi	21.4	23	22	22	21	28	22
	35.0	39	38	21	20	32	33
	45.6	54	49	22	21	47	39
	52.5	67	63	21	20	46	44
30 psi	21.4	23	21	21	17	22	20
	35.0	38	38	21	20	33	28
	45.6	55	46	21	17	40	34
	52.5	67	61	21	20	47	41

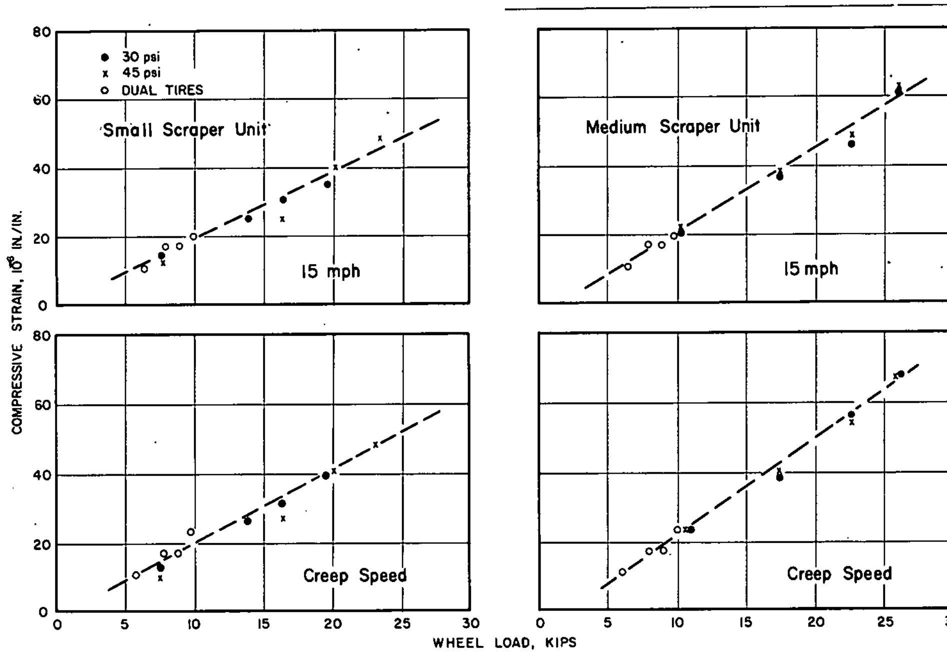


Figure 44. Relationship between wheel load and compressive strain, Section 397 (11-6 design).

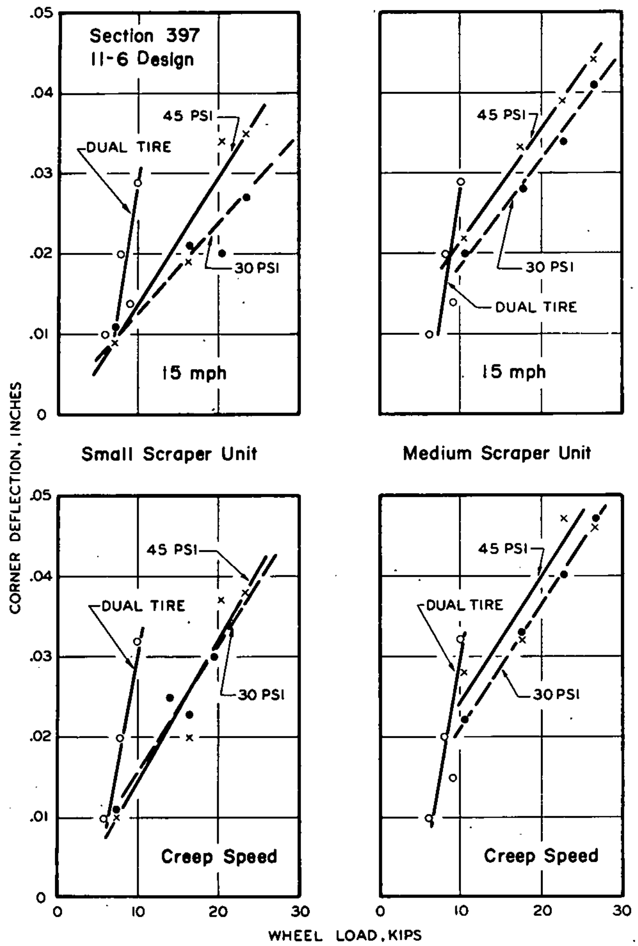


Figure 45. Effect of tire pressure on corner deflection.

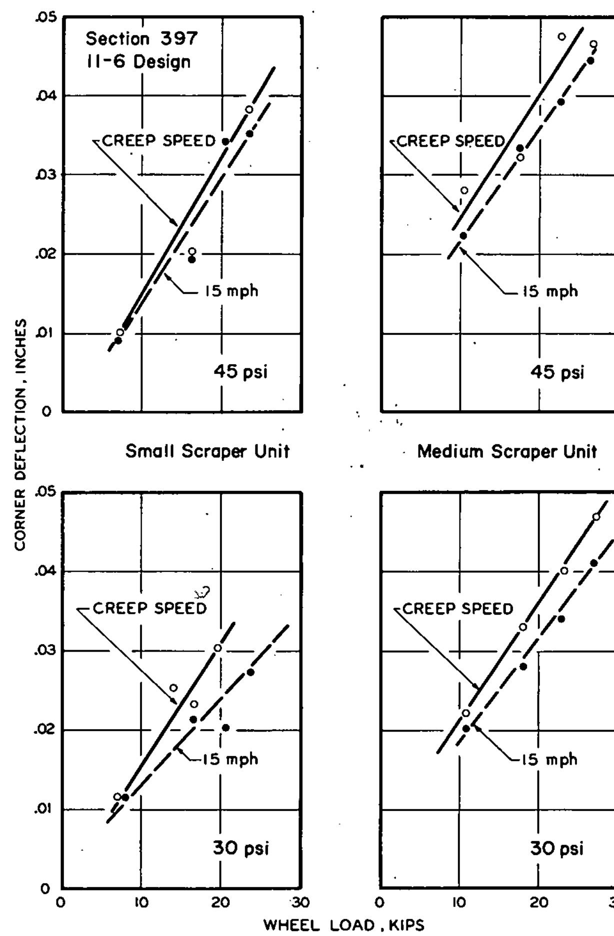


Figure 46. Effect of vehicle speed on corner deflection.

TABLE 43

MEAN DYNAMIC LOAD EFFECT

Vehicle	Axle Load (kips)	Tire Size (in.)	Section	Load Effect (lb): 10 Mph, 30 Psi	10 Mph, 45 Psi	20 Mph, 30 Psi	20 Mph, 45 Psi
Small scraper	39.6	26.5x25	Rough	3,420	7,580	23,410	13,370
			Smooth	2,620	4,340	4,170	9,080

TABLE 44

PERCENT CHANGE IN AXLE LOAD OVER STATIC LOAD

Vehicle	Axle Load (kips)	Tire Size (in.)	Section	Change (%): 10 Mph, 30 Psi	10 Mph, 45 Psi	20 Mph, 30 Psi	20 Mph, 45 Psi
Small scraper	29.6	26.5x25	Rough	8.6	19.1	59.1	33.6
			Smooth	6.6	11.0	10.5	22.9

4.7 BRIDGE STUDY

The two scraper units and a number of conventional tractor-semitrailer vehicles (axle load range from 12-kip single to 40-kip tandem) were operated over the five bridge structures in Loop 6 at speeds of creep and 15 mph. Dynamic measurements of maximum midspan strain and deflection for each of three beams were taken with the vehicle symmetrically located with respect to the three beams.

The strains and deflections at 15 mph (maximum safe speed for scraper units) are expressed in terms of their relationship to creep speed strain and deflection as described in Chapter 3, Section 3.7.

The limited scope of the study restricted comparisons to the mean amplification factors for the three classes of vehicles (single axle, tandem axle and scraper units) for each structure and for all structures at one speed.

The mean strain and deflection amplification factors for each of the bridge structures in the study (Table 39) are given in Table 45. The vehicles were separated into classes, and the mean amplification factors by vehicles and classes were computed.

The mean amplification factors by vehicle classes show that the factors for the tandem axle vehicles were slightly lower than those for the single axle vehicles as reported in Chapter 3. Furthermore, the mean amplification factors for the scraper units fell between those for the single and tandem axle vehicles. However, the trend of the mean factors by vehicle classes for each bridge, did not agree entirely with the trends for the mean factors for all bridges.

The mean factors for each bridge show that the strain factor for the medium scraper was greater than for the small scraper. No consistent trend existed for the deflection amplification factors for these units.

The effect of vehicle speed on the amplification factor cannot be determined from this study.

4.8 NEEDED RESEARCH

The findings and trends established in this study were generally conclusive, but the restric-

TABLE 45

MEAN AMPLIFICATION FACTORS FOR ALL BRIDGES, LOOP 6, 15 MPH

Vehicle Class	Axle Load (kips)	Factor for Bridge Indicated:					
		3B	8A	8B	9A	9B	Mean
(a) Strain Amplification Factor							
Single axle	12 S	1.117	1.083	1.036	1.030	1.034	1.060
	18 S	1.127	1.142	1.131	1.020	1.071	1.098
Tandem axle	32 T	1.041	1.121	1.085	1.000	1.044	1.058
	40 T	1.026	1.075	1.100	1.000	1.048	1.049
Scraper unit	Small [1]	1.019	1.126	1.031	1.028	1.063	1.053
	Medium [2]	1.065	1.129	1.096	1.016	1.035	1.068
Mean:							
Singles		1.122	1.113	1.084	1.025	1.053	1.079
Tandems		1.033	1.098	1.093	1.000	1.046	1.054
Scrapers		1.042	1.128	1.064	1.022	1.049	1.061
(b) Deflection Amplification Factor							
Single axle	12 S	1.103	1.102	1.076	1.019	1.040	1.068
	18 S	1.075	1.142	1.163	1.000	1.144	1.095
Tandem axle	32 T	1.033	1.129	1.108	1.018	1.081	1.073
	40 T	1.055	1.077	1.090	1.044	1.093	1.071
Scraper unit	Small [1]	1.000	1.059	1.035	1.037	1.103	1.046
	Medium [2]	1.131	1.122	1.150	1.039	1.122	1.112
Mean:							
Singles		1.089	1.122	1.120	1.010	1.092	1.082
Tandems		1.044	1.098	1.099	1.031	1.087	1.072
Scrapers		1.066	1.091	1.093	1.038	1.113	1.079

[1] Axle loads, 47.3 and 40.7 kips.

[2] Axle loads, 61.2 and 48.8 kips.

tions on vehicle speed, pavement designs and instrument locations make further research desirable.

Dynamic measurements of strain and deflection in rigid pavement sections should be made at locations other than the edge or corner of the pavement surface. Transmitted pressure instrumentation at other levels within flexible pavement structures may indicate greater effects of vehicle speed and/or tire inflation pressure.

An expansion of the dynamic load-tire pressure study to include both axles of the unit, impact and acceleration tests might establish other trends not detected in this program.

Based on the findings of the main Road Test, a performance study of these units versus equivalent axle loads on conventional units would be desirable.

Chapter 5

Special Suspension Systems

5.1 SUMMARY

The objective of this study was to investigate the dynamic effects on pavements and bridges of vehicles equipped with special suspension systems and to compare these to the dynamic effects of conventional vehicles with similar axle loads and tire pressures.

The instrumentation available on the Road Test at the time of this study made possible the measurement of strain, deflection, transmitted embankment pressure and dynamic load. The tests were conducted on existing pavement sections under the procedure described in the following sections.

Difficulty in determining exact vehicle speed and transverse location of the vehicles at the instrument locations caused considerable scatter in the data which in turn necessitates the reporting of general trends rather than conclusive findings.

Flexible Pavement Study

The changes in deflection (total, embankment and structure) for the several designs of suspension systems within the limits of this study were less than the differences attributed to experimental error. An increase in vehicle speed caused a comparable decrease in pavement deflection for all special suspension and conventional vehicles.

The pressure transmitted to the embankment soil was generally lower for the special suspension units than for the conventional units with little variation apparent among the several units. The effect on the transmitted pressure of vehicle speed was uniform for all special suspension and conventional units, and the transverse or longitudinal distribution of the transmitted pressure showed no effect of the design of the suspension systems.

Rigid Pavement Study

The decrease in edge strain and corner deflection caused by an increase in vehicle speed was reasonably uniform for the several special suspension systems. Similar relationships for the conventional units showed no appreciable differences which could be associated with changes in the suspension systems.

Dynamic Load Study

For the two sections of pavement tested, the dynamic load effect for all units was greater for the pavement with lower serviceability and also increased with increase in vehicle speed. The relative dynamic load effect of the several suspension systems indicated some variation subject to vehicle speed and pavement serviceability.

Bridge Study

The non-uniform loading of the test vehicles coupled with other vehicle characteristics not determined in this study negate the findings to some degree.

In general, however, the mean amplification factors (strain and deflection) for the conventional single axle vehicles were higher than those for the tandem axle vehicles, both conventional and special. This finding agrees with those given in Road Test Report 4.

The mean strain amplification factors for all the special vehicles were appreciably higher than the factors for the conventional units at 30 mph. At speeds of 15 mph, this was not true for either the strain or deflection amplification factors.

5.2 SCOPE

To evaluate the effect of different suspension systems on the pavement, bridges and cargoes, a program was conducted including measurements of dynamic strains and deflections, pressures transmitted to the embankment soil, accelerations in the vehicle, and tire pressure-dynamic load relationships. All of these relationships are discussed in this chapter except the vehicle and cargo accelerations which are grouped for all vehicles and studies in Chapter 8.

5.3 DESCRIPTION OF MEASUREMENTS

Three tractors and two semitrailers equipped with special suspension systems were made available to the Road Test by the manufacturers through the efforts of the Department of Army, the Automobile Manufacturers Association and the Truck Trailer Manufacturers' Association.

The types of suspension investigated were determined largely on the basis of their availability. Table 46 gives the vehicles included in the study, their characteristics, and a brief description of their suspension systems; and also the characteristics of vehicles equipped with conventional suspension systems and LPLS tires. These latter units were used for

TABLE 46

VEHICLE CHARACTERISTICS

Vehicle	Axle Load (kips)	Tire Size (in.)	Tire Press. (psi)	Gross Load (kips)	Axle Load (kips)					Axle Spacing (in.)				Center of Duals (in.)	Gross Tire Contact Area[1] (sq in.)
					1	2	3	4	5	1–2	2–3	3–4	4–5		
61[2]	32 T	9.00x20N	75	50.4	8.2	5.4	5.3	15.5	16.0	144	53	227	52	72	523.6
64[3]	32 T	11.00x20N	75	50.7	8.4	5.1	5.0	16.3	15.9	144	52	215	58		461.7
65[4]	32 T	9.00x20N	80	71.8	8.4	15.8	16.5	15.6	15.5	102	50	234	54	61	442.8
66[5]	32 T	11.00x20N	75	70.6	7.2	15.9	15.6	14.8	17.1	121	52	236	52	71	
67[6]	32 T	10.00x20N	75	71.1	7.4	16.0	15.7	15.3	16.7	121	52	238	52	71	
Conv.	12 S	7.50x20N	80	28.1	3.7	12.4	12.0			142	252			69	181.5
Conv.	18 S	10.00x20N	80	40.3	5.6	16.8	17.9			143	246.5			72	276.6
Conv.	32 T	9.00x20N	80	74.5	9.5	16.4	16.1	15.4	17.1	129	48	241	50	72	443.9
Conv.	40 T	11.00x20N	80	90.2	9.0	20.4	20.0	20.7	20.1	144	50	241	50	72	630.4
LPLS[7]	32 T	46.00x24	35	71.2	9.6	15.1	15.1	15.3	16.1	87	54	247	54	72	
Conv.	24 T	7.50x20N	80	55.4	5.8	12.2	12.0	12.3	13.1	135	48	232	50	70	358.5
Conv.	22.4 S	11.00x20N	80	51.3	6.2	22.7	22.4			137	246			71	230.8

[1] Per axle or axles.
[2] Vehicle 61 furnished by Hutchens and Son Metal Products Company; a tandem axle conventional semitrailer equipped with combination fluid and air suspension system.
[3] Vehicle 64 furnished by Hutchens and Son Metal Products Company; a semitrailer equipped with unique staggered wheel suspension system of which no axle is common to any two wheels.
[4] Vehicle 65 furnished by White Motor Company; a model 3400 TD tandem tractor equipped with variable single-leaf spring suspension system with rear axle drive.
[5] Vehicle 66 furnished by International Harvester Company; a model VF–195 tandem tractor equipped with standard Hendrickson walking beam type suspension with rubber load cushions.
[6] Vehicle 67 furnished by International Harvester Company; a model VF–195 tandem tractor equipped with standard Hendrickson walking beam type suspension with steel-leaf springs. (This unit was used for comparative purposes only.)
[7] LPLS (H–2) M–52 tractor semitrailer equipped with low-pressure, low-silhouette tires.

TABLE 47

TOTAL DEFLECTION VALUES FOR FLEXIBLE SECTIONS

Deflection (10^{-3} in.)

(a) SECTION 265, 5-9-16 DESIGN

Vehicle	Axle Load (kips)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	OWP Mean	IWP Mean	Section Mean
61	32 T	14	11	13	14	11	11	13	12	13
64		13	10	10	12	9	8	11	10	11
65		12	12	12	14	12	10	12	12	12
66		15	12	11	15	11	10	13	12	12
67		14	12	12	14	11	10	13	12	12
Conv.	40 T	18	15	15	18	13	13	16	15	15
Conv.	12 S	11	9	8	10	7	6	9	8	9
LPLS	32 T	15	12	10	15	12	9	12	12	12
Conv.	18 S	15	11	12	14	11	10	13	12	13
Conv.	32 T	15	13	13	12	11	9	14	11	13

(b) SECTION 333, 6-9-16 DESIGN

Vehicle	Axle Load (kips)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	OWP Mean	IWP Mean	Section Mean
61	32 T	24	18	17	20	13	13	20	15	18
64		18	13	11	11	9	8	14	9	12
65		21	19	16	16	14	11	19	14	16
66		24	19	17	21	14	13	20	16	18
67		24	20	17	20	14	12	20	15	18
Conv.	40 T	28	23	21	21	15	13	23	16	20
Conv.	12 S	18	17	13	14	11	8	16	11	14
LPLS	32 T	26	22	20	17	14	12	23	14	19
Conv.	18 S	24	22	18	20	16	13	21	18	19
Conv.	32 T	20	17	15	14	11	10	17	12	15

(c) SECTION 271, 6-9-8 DESIGN

Vehicle	Axle Load (kips)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	OWP Mean	IWP Mean	Section Mean
61	32 T	32	23	15	24	17	12	22	18	20
64		26	20	16	17	13	10	21	13	17
65		28	22	19	24	22	14	23	20	21
66		33	28	21	24	20	15	28	20	24
67		32	27	24	25	20	17	28	21	24
Conv.	40 T	42	35	21	29	25	19	33	24	28
Conv.	12 S	26	19	14	16	11	9	20	12	16
LPLS	32 T	32	27	19	23	19	13	26	18	22
Conv.	18 S	31	27	23	22	19	16	27	19	23
Conv.	32 T	31	24	20	21	17	13	25	17	23

(d) SECTION 301, 6-6-16 DESIGN

Vehicle	Axle Load (kips)	Outer Wheelpath Creep	Outer Wheelpath 15 Mph	Outer Wheelpath 30 Mph	Inner Wheelpath Creep	Inner Wheelpath 15 Mph	Inner Wheelpath 30 Mph	OWP Mean	IWP Mean	Section Mean
61	32 T	17	13	15	16	11	13	15	13	14
64		14	11	9	11	10	8	11	10	10
65		15	13	13	13	12	12	14	12	13
66		16	13	12	14	12	11	14	12	13
67		17	14	13	16	12	11	15	13	14
Conv.	40 T	20	16	14	17	14	13	17	15	16
Conv.	12 S	13	10	9	10	7	6	11	8	10
LPLS	32 T	19	15	15	16	13	13	16	14	15
Conv.	18 S	17	14	13	15	12	11	15	13	14
Conv.	32 T	16	12	11	12	10	10	13	11	12

comparison. Each of the special suspension vehicles is numbered.

The train of vehicles was operated over pavement sections in Loop 6 equipped with electronic devices for measuring deflection and strain, and in Loop 4 equipped with electronic devices for measuring embankment soil pressures, and over bridge structures of different designs on Loop 6 equipped with devices to measure midspan strain and deflections. Table 39 lists the sections and structures included in the test with notations of the measurements taken.

The sections selected for this study were the thicker designs remaining in each of the two loops. Furthermore, sections which had experienced little or no distress during the regular test were selected.

The vehicles were operated over each of these sections and bridges in random order at three levels of speed (creep, 15 and 30 mph). At least four passes within a prescribed transverse placement were required for each vehicle at each speed level. The tests were conducted, as far as practical, in only one flexible and one rigid section per day to reduce the temperature effect. A section-to-section analysis (either rigid or flexible) was not practical because of wide temperature fluctuations during the study.

In addition to the measurements of dynamic strain and deflection, the dynamic load applied

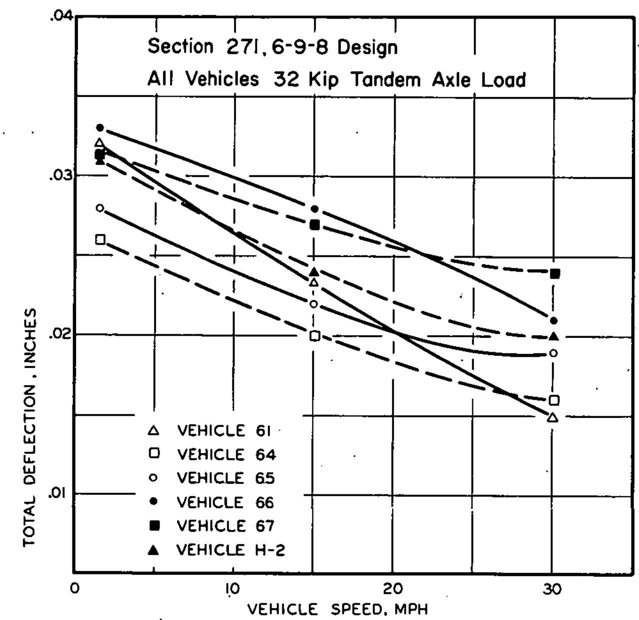


Figure 47. Relationship between deflection and vehicle speed.

TABLE 48

MEAN DEFLECTION SUMMARY OF FLEXIBLE SECTIONS, 32-Kip Tandem Axle Load

Vehicle	Deflection (10^{-3} in.) Sect. 265 (5–9–16)	Sect. 301 (6–6–16)	Sect. 333 (6–9–16)	Sect. 271 (6–9–8)	Mean
61	13	14	18	17	16.3
64	11	10	12	21	12.5
65	12	13	16	24	15.5
66	12	13	18	24	16.8
67	12	14	18	24	17.0
LPLS	12	15	19	22	17.0
Conv.	13	12	15	21	15.3

TABLE 49

TRANSMITTED EMBANKMENT PRESSURES, SECTION 581 (5–6–12 DESIGN)

Vehicle	Axle Load (kips)	Tire Size (in.)	Tire Press. (psi)	Zero Reading[1] (in.) Creep	15 Mph	30 Mph	Pressure at Embankment (psi) Creep	15 Mph	30 Mph	Zero Reading[2] (in.) Creep	15 Mph	30 Mph	Mean Zero[1] Reading (in.)	Mean Press. (psi)	Mean Zero[2] Reading (in.)
61	32 T	9.00x20N	75	41	45	46	4.73	3.85	3.32	73	70	72	44	3.97	72
64		11.00x20N	75	45	50	60	3.15	2.75	2.50	52	58	58	52	2.80	56
65		9.00x20N	80	35	40	70	4.62	4.10	3.15	50	75	40	48	3.96	55
66		11.00x20N	75	40	55	52	4.60	4.24	3.79	57	95	84	49	4.21	79
67		10.00x20N	75	40	45	47	4.62	4.05	3.92	58	60	57	44	4.20	58
LPLS		46.00x24	35	49	55	50	4.20	4.20	3.40	78	85	85	51	3.93	83
Conv.	12 S	7.50x20N	80	40	50	38	4.20	3.40	3.30	46	60	46	43	3.63	51
Conv.	24 T	7.50x20N	80	48	45	36	4.40	3.25	3.40	54	60	56	43	3.68	57
Conv.	18 S	10.00x20N	80	52	50	46	5.70	4.20	4.50	60	60	43	49	4.80	54
Conv.	32 T	9.00x20N	80	35	49	40	5.00	4.22	4.25	68	63	33	41	4.49	55
Conv.	22.4 S	11.00x20N	80	54	55	52	6.80	5.30	5.30	60	65	60	55	5.80	62

[1] Leading distance to 0 pressure.
[2] Trailing distance to 0 pressure.

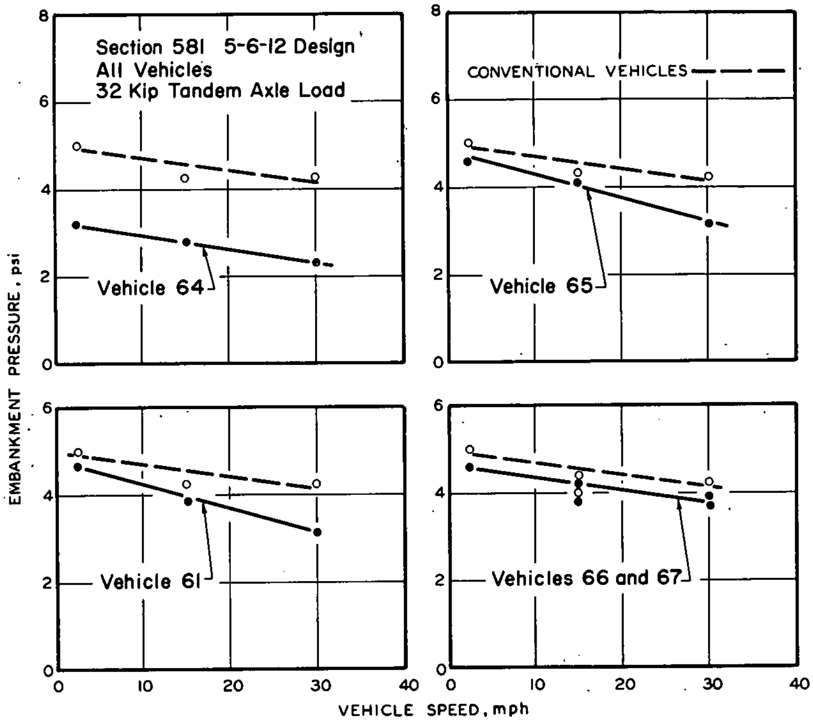


Figure 48. Relationship between vehicle speed and embankment pressure.

to the pavement was studied. A description of the measuring system was given in Chapter 3.

The data accumulated in the study of the special suspension vehicles are presented and discussed as follows: first, relationships for the flexible sections; second, relationships for the rigid sections; third, the discussion of the tire pressure-dynamic load relationships; and, fourth, the relationships for the bridge structures.

5.4 FLEXIBLE PAVEMENT STUDY

The experiment was designed so that the relationships could be developed between vehicle speed and deflection, embankment pressure and deflection, embankment pressure and vehicle speed, and between embankment pressure and vehicle placement. In addition, comparisons of these relationships with those for conventional vehicles could be shown.

Table 47 gives the mean total deflection under each vehicle for the four flexible sections included in the study. The embankment and structural deflection tests indicated that similar relationships existed at all levels.

The deflections shown for vehicle 64 are generally lower than the deflections for the other vehicles. This can be explained by the arrangement of the wheels and axles on this unit. No two wheels are combined into a dual unit and no two wheels have a common axle. Thus, the deflections are produced not by a dual tire load of 8-kip (32-kip tandem axle load) but by a single wheel load of 4 kip.

The deflections for vehicles 66 and 67 are essentially the same. The units are identical in design except for the suspension (Table 46).

Figure 47 shows the relationship between vehicle speed and deflection for the suspension system vehicles and a conventional unit. The relationships are normal in that deflection decreases exponentially with increase in speed and indicate no apparent effect of the suspension system design other than that for vehicle 64.

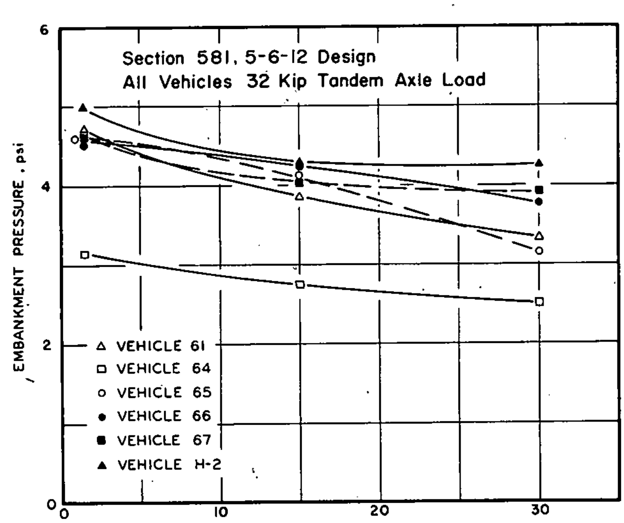


Figure 49. Relationship between vehicle speed and embankment pressure.

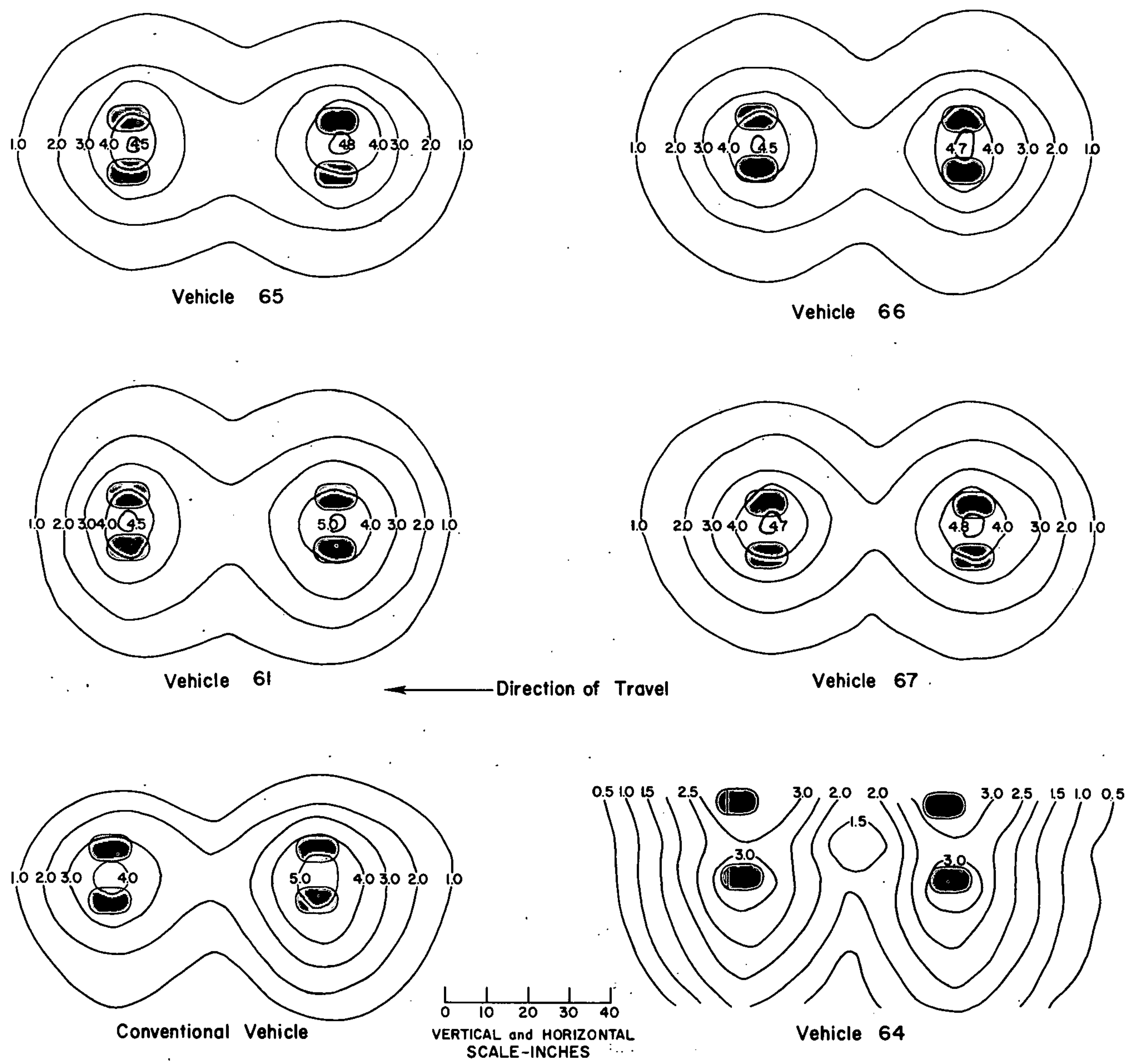


Figure 50. Embankment pressure influence diagrams, creep speed, Section 581 (5-6-12 design).

Table 48 gives the mean deflections for the four sections for each vehicle (three speed levels and both wheel paths). The range of the mean deflections recorded in any one section was less than 0.005 in. (excluding vehicle 64) and the range of the mean deflections for all vehicles for the four sections was less than 0.002 in. No consistent trend relating to the design of the suspension systems was indicated in the deflections for any of the four sections.

The mean pressures transmitted to the embankment soil are given in Table 49. The distances from the loaded axle, leading and trailing, at which a zero pressure was recorded are also given. In addition, the mean values of pressure and distance are shown.

Figure 48 shows the relationship of transmitted pressure and vehicle speed for each of the vehicles with special suspension systems as well as the relationship of vehicle speed and embankment pressure for a conventional unit. The uniformity of the relationships for vehicles 66 and 67 is apparent. In all comparisons the transmitted pressure is greater at all speeds for the conventional unit than for the special suspension units, and the change of pressure with vehicle speed is about the same for all units.

Figure 49 combines the vehicle speed and transmitted pressure relationships for all special suspension vehicles as compared to the conventional unit. The distance from the loaded axle at which a zero pressure was recorded shows no discernible trend associated with different suspension systems. The increase in distance with an increase in speed noted in other tests exists for the distances recorded ahead of the loaded axle but does not exist for the distances recorded behind the loaded axle.

Pressure influence diagrams developed from the pressure distribution study are shown in Figure 50. The position of the loaded dual or loaded single wheels are shown relative to the gage point. The diagrams for vehicles 61, 65, 66 and 67 and the conventional vehicle show little effect of the different suspension system designs. However, the diagram for vehicle 64 shows the effect of the individual suspended wheels. The maximum transmitted pressure is less under the single wheel load and the effect of the adjacent wheel is clearly evident.

The influence diagrams for these units at other than creep speed vary only in the magnitude of the transmitted pressures with a slight elongation of the influence lines with increase in speed. Data from which these diagrams were prepared can be found in Road Test Data System No. 9169.

5.5 RIGID PAVEMENT STUDY

The five vehicles with special suspension systems plus a number of conventional units were operated over four rigid pavement sections in Loop 6. Dynamic measurements of strain and deflection were recorded during tests at three levels of speed.

Table 50 lists mean values of strain (tensile and compressive) and deflection for all the vehicles in the study for each of the four sections. Each value is the mean of at least four field recordings taken when the vehicles were at a prescribed transverse placement.

The relationships between vehicle speed and compressive strain and deflection are shown in Figures 51 and 52 for section 389. Relationships for a conventional unit (also shown on each plot) compare well with those for the suspension units with the exception of vehicle 64 (Figure 52). This vehicle was discussed in Section 5.4.

Relationships as shown could be developed for each of the other three rigid sections. The data (Data System No. 9253) show no appreciable differences between these relationships and those for the other sections included in the study.

5.6 DYNAMIC LOAD STUDY

The instrumentation described in Chapter 3 (Section 3.6) was adapted for use with the vehicles and axle loads in this group. The vehicles were operated over the two sections of pavement on Loop 6 at speeds of 10 and 30 mph. The serviceability indexes of the sections were 2.70 and 4.66 and are generally classified for this study as rough and smooth, respectively.

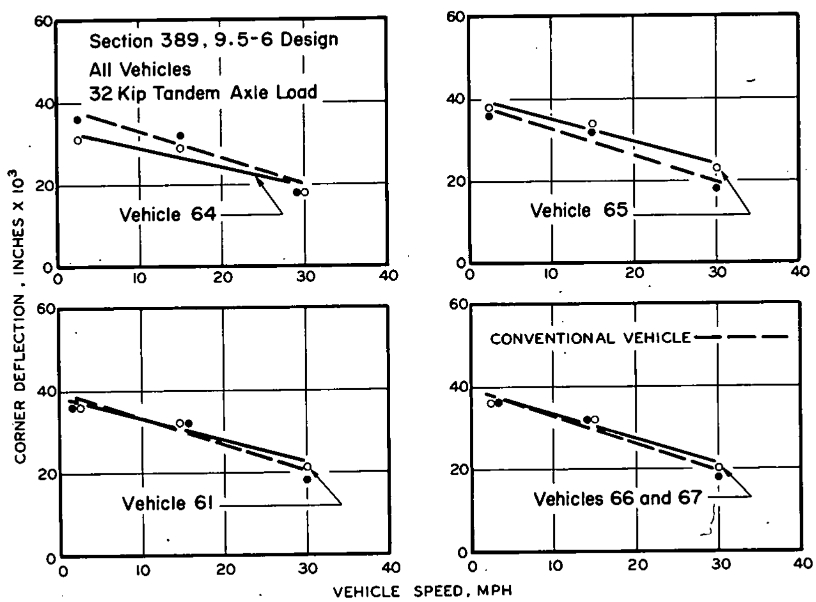


Figure 51. Relationship between corner deflection and vehicle speed.

TABLE 50

STRAIN AND DEFLECTION VALUES FOR RIGID SECTIONS

Vehicle	Axle Load (kips)	Tire Pressure (psi)	Strain (10^{-6} in./in.) Compressive Creep	Strain Compressive 15 Mph	Strain Compressive 30 Mph	Strain Tensile Creep	Strain Tensile 15 Mph	Strain Tensile 30 Mph	Deflection (10^{-3} in.) Creep	Deflection 15 Mph	Deflection 30 Mph	Mean Strain (10^{-6} in./in.) Compressive	Mean Strain Tensile	Mean Deflection (10^{-3} in.)
(a) SECTION 359, 12.5R–3 DESIGN														
61	32 T	75	14	12	13	8	6	6	13	10	8	13	7	10
64		75	11	12	11	7	6	5	11	10	9	11	6	10
65		80	16	13	12	9	8	7	13	11	10	14	8	11
66		75	13	12	12	9	8	8	12	10	11	12	8	11
67		75	13	11	12	11	9	8	12	10	11	11	9	11
LPLS		35	13	12	11	9	8	8	12	10	10	12	8	11
Conv.	40 T	80	18	16	14	11	10	9	16	14	12	16	10	14
Conv.	12 S	75	9	7	7	4	4	3	5	4	4	8	4	4
Conv.	18 S	80	14	14	12	7	6	5	8	7	6	13	6	7
Conv.	32 T	80	14	14	11	9	8	7	12	11	10	13	8	11
(b) SECTION 389, 9.5–6 DESIGN														
61	32 T	75	20	18	15	14	13	11	36	32	21	18	13	30
64		75	13	13	13	13	13	8	31	29	18	13	11	26
65		80	20	18	19	15	14	10	38	34	23	19	13	32
66		75	18	18	17	16	15	10	36	32	20	19	14	29
67		75	18	16	15	16	15	10	36	32	20	16	14	29
LPLS		35	22	19	19	19	17	13	42	38	24	20	6	35
Conv.	40 T	80	15	12	12	10	9	6	20	17	10	13	8	16
Conv.	12 S	75	19	17	19	21	20	12	35	33	20	18	18	29
Conv.	18 S	80	21	19	17	14	14	11	27	24	17	19	13	22
Conv.	32 T	80	19	17	16	16	14	10	36	32	18	17	13	25
(c) SECTION 381, 9.5R–3 DESIGN														
61	32 T	75	17	15	14	9	8	8	18	14	11	12	8	14
64		75	15	12	10	8	8	8	17	13	11	12	8	10
65		80	17	18	14	9	9	9	19	17	12	17	9	16
66		75	18	13	13	10	9	9	19	13	12	15	9	15
67		75	18	15	12	9	9	8	18	15	11	15	9	15
LPLS		35	22	19	17	13	12	10	24	21	16	19	12	20
Conv.	40 T	80	12	11	8	5	5	4	9	8	5	10	5	7
Conv.	12 S	75	17	14	14	12	11	9	15	15	11	15	11	14
Conv.	18 S	80	17	16	13	8	7	7	13	10	8	15	7	10
Conv.	32 T	80	19	16	13	9	8	8	19	15	13	14	8	16
(d) SECTION 397, 9.5–6 DESIGN														
61	32 T	75	18	17	16	10	9	8	19	15	16	17	9	17
64		75	14	14	12	10	9	7	16	14	12	13	9	14
65		80	19	18	16	11	10	8	19	17	16	18	10	17
66		75	19	17	16	11	10	9	19	17	15	17	10	17
67		75	20	18	16	12	11	8	19	17	16	18	10	17
LPLS		35	17	17	14	15	13	13	18	16	16	16	14	17
Conv.	40 T	80	23	21	20	15	13	11	24	20	18	21	13	21
Conv.	12 S	75	13	13	11	6	6	4	10	8	6	12	5	8
Conv.	18 S	80	18	18	17	9	8	7	13	12	11	18	8	12
Conv.	32 T	80	18	17	16	11	10	9	19	16	16	17	10	17

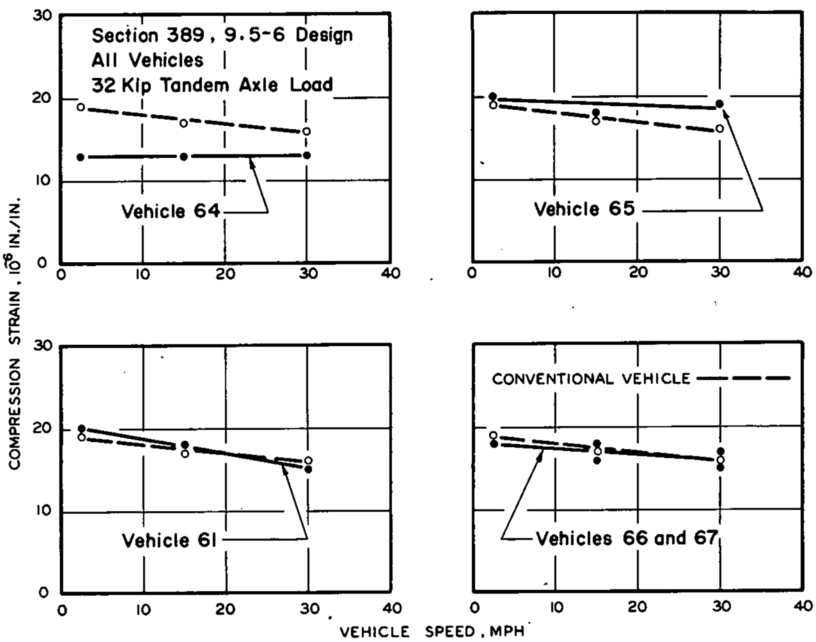


Figure 52. Relationship between vehicle speed and compressive strain.

Table 51 gives the mean dynamic load effect for each of the special suspension vehicles, the conventional unit, and the vehicle equipped with LPLS tires for both sections at two levels of speed. The total dynamic load effect for the tandem vehicles would be twice that shown.

The relationships noted for other vehicles between dynamic load effect and vehicle speed and between dynamic load effect and initial pavement serviceability also exist for these data. Increase in vehicle speed and a lower pavement serviceability caused an increase in dynamic load effect. The effect of the different suspension systems on the dynamic load effect was not consistent at the various levels of speed and serviceability studied.

Ratios of dynamic load effect for all special vehicles to the dynamic load effect for the conventional unit are given in Table 52. The ratios for vehicles 66 and 67 were reasonably consistent and showed a beneficial effect of the special suspension system on vehicle 66. For

TABLE 51

MEAN DYNAMIC LOAD EFFECT
(All Vehicles 32-Kip Tandem Axle Loads)

Vehicle [1]	Tire Size (in.)	Tire Pressure (psi)	Tire Contact Area per Axle (sq in.)	Load Effect (lb)			
				Rough Pavement		Smooth Pavement	
				10Mph	30Mph	10Mph	30Mph
61	9.00x20	75	272.8	1,360		1,530	4,490
64	11.00x20	75	228.4[2]	1,190	3,160	1,170	2,520
65	9.00x20	80	233.2	1,210	2,680	990	1,930
66	11.00x20	75	291.2	1,030	1,370	840	1,220
67	10.00x20	75	280.4	1,530	2,380	1,110	1,870
LPLS	46.00x24	35		1,200	4,940	780	1,600
Conv.	9.00x20	80	234.8	1,710	2,320	1,140	1,960

[1] All vehicles, 32-kip tandem axle loads.
[2] Total contact area for four external tires.

TABLE 52

RATIO OF DYNAMIC LOAD EFFECT OF SUSPENSION VEHICLES TO CONVENTIONAL UNIT

Vehicle	Tire Size (in.)	Tire Press. (psi)	Tire Contact Area per Axle (sq in.)	Ratio: Rough Pavement 10Mph	Ratio: Rough Pavement 30Mph	Ratio: Smooth Pavement 10Mph	Ratio: Smooth Pavement 30Mph
61	9.00x20N	75	272.8	0.80		1.34	2.29
64	11.00x20N	75	228.4 [1]	0.70	1.36	1.03	1.29
65	9.00x20N	80	233.2	0.71	1.16	0.87	0.98
66	11.00x20N	75	291.2	0.60	0.59	0.74	0.62
67	10.00x20N	75	280.4	0.89	1.02	0.97	0.95
LPLS	46.00x24	35		0.70	2.10	0.68	0.82
Conv.	9.00x20N	80	234.8	1.00	1.00	1.00	1.00

[1] Total contact area for four external tires.

TABLE 53

MEAN AMPLIFICATION FACTORS, ALL BRIDGES, LOOP 6

Vehicle	Axle Load (kips)	3B 15 Mph	3B 30 Mph	8A 15 Mph	8A 30 Mph	8B 15 Mph	8B 30 Mph	Mean 15 Mph	Mean 30 Mph
(a) STRAIN AMPLIFICATION FACTOR									
61	32 T	1.015	1.269	1.026	1.106	1.099	1.148	1.046	1.174
64	32 T	1.075	1.060	1.070	0.964	1.048	1.134	1.064	1.052
65	32 T	0.973	1.157	1.037	1.045	1.032	....	1.014	1.101
66	32 T	1.027	1.041	1.098	1.121	1.058	1.058	1.061	1.073
67	32 T	0.962	1.051	1.083	1.121	1.066	1.066	1.037	1.079
LPLS	32 T	1.024	1.195	1.108	1.162	1.095	1.110	1.075	1.155
Conv.	32 T	1.039	1.026	1.060	1.045	1.057	1.041	1.052	1.037
Conv.	40 T	1.043	1.097	1.005	1.005	1.013	1.006	1.020	1.036
Conv.	12 S	1.114	1.114	1.081	1.049	1.090	1.163	1.095	1.108
Conv.	18 S	1.120	1.160	1.022	1.055	1.100	....	1.080	1.107
Mean, conventional tandems								1.036	1.037
Mean, conventional singles								1.088	1.108
(b) DEFLECTION AMPLIFICATION FACTOR									
61	32 T	1.108	1.456	1.044	1.149	1.101	1.115	1.100	1.240
64	32 T	1.106	1.085	1.102	1.014	1.072	1.130	1.093	1.076
65	32 T	1.049	1.278	1.072	1.120	1.116	....	1.079	1.119
66	32 T	1.050	1.111	1.132	1.228	1.116	1.081	1.099	1.140
67	32 T	1.032	1.147	1.059	1.214	1.103	1.057	1.056	1.109
LPLS	32 T	1.015	1.242	1.072	1.145	1.113	1.195	1.066	1.194
Conv.	32 T	1.050	1.152	1.132	1.180	1.149	1.068	1.110	1.133
Conv.	40 T	1.040	1.173	1.120	1.170	1.085	1.066	1.081	1.136
Conv.	12 S	1.137	1.172	1.075	1.050	1.095	1.166	1.102	1.129
Conv.	18 S	1.073	1.219	1.070	1.157	1.101	....	1.081	1.188
Mean, conventional tandems								1.096	1.135
Mean, conventional singles								1.092	1.159

vehicles 61 and 64, with the exception of the 10 mph run on rough pavement, the ratio indicated a greater dynamic load effect under the special suspension than under the conventional vehicle.

These observations cannot be considered conclusive since the speed and transverse position of the vehicle appeared to be quite critical and since these variables were very difficult to control.

5.7 BRIDGE STUDY

The vehicles described previously in this chapter were operated over Bridges 3B, 8A and 8B at speeds of creep, 15 and 30 mph. Measurements of maximum midspan strain and deflection on the three beams were taken with the vehicles positioned symmetrically on the bridge. The objective of this portion of the study was to determine the effect, if any, of the various suspension systems on the dynamic measurements taken.

Table 46 gives the characteristics of each of the vehicles, including certain conventional vehicles and an M-52 tractor-semitrailer equipped with LPLS tires. Two of the special suspension systems were on semitrailer axles and three were on tractor or drive axles.

The characteristics of these units (axle and gross loads and spring constants, for example) could not be determined from the data at hand. This must be kept in mind in reviewing the data for possible effects of the suspension systems on the dynamic measurements.

Table 53 gives the mean amplification factors (Chapter 3, Section 3.7) for each of the vehicles for the three structures tested. The mean factors for all bridges and for all classes of vehicles (conventional only) are shown. Higher amplification factors were associated with the 30 mph speeds for all vehicles (as previously reported in Chapter 3) except for vehicle 64. Although the differences between the 15 and 30 mph factors for this unit were slight, there was a definite reversal of the expected effect.

The mean amplification factors for the conventional single axle vehicles were found to be higher than those for the conventional tandem axle units, with the exception of the 15 mph deflection amplification factors.

For the special suspension systems, the mean strain amplification factors at 15 mph were similar to those for the conventional tandems, but the factors at 30 mph were appreciably higher for the special suspension vehicles with the exception of vehicle 64 which was only slightly higher. However, for the deflection amplification factors, the means for both the conventional and special vehicles showed no consistent trend.

Although the differences are slight, the mean factors for vehicle 61 are generally higher than those for the other special suspension system units at both speeds.

5.8 NEEDED RESEARCH

The instrumentation available at the Road Test was primarily designed to detect variations caused by axle load, vehicle speed and transverse placement, and thus, was not capable of detecting the apparently small differences associated with changes in vehicle suspension systems. In addition, the amplification factors used in comparing the effects of the various units on the bridges are dependent upon gross vehicle loads and other vehicle characteristics such as spring and tire constants which were not uniform nor determined for these units.

Research to determine the relative effect of the various designs of suspension systems should be directed first to the study of the vehicle and then to the study of the effect of new suspensions systems on the pavements and bridges.

Chapter 6

Military Vehicles, Tire

6.1 SUMMARY

The objective of this study was to investigate the dynamic effects on pavements and bridges of specialized units of military highway and off-highway equipment and to compare these effects, where possible, with those for conventional units at several axle loads and vehicle speeds.

A group of vehicles (described in detail in the following sections) were operated over pavement sections equipped with instrumentation to measure the dynamic strains and deflections, embankment pressures and dynamic wheel loads, and over test structures equipped with instruments to detect maximum strains and deflection.

As was true for the other special studies, the large number of indeterminate variables and vehicle characteristics limit the findings of this program to general trends and indicate that further research might be productive.

Flexible Pavement Study

For all the military units in this study the effect of vehicle speed on the deflection and transmitted embankment pressure agreed with the relationships found for conventional units.

For two of the heavy duty transporters (GOER and HETAG) the rate of increase of pavement deflection with increase in wheel load was greater than for the conventional units.

Individual relationships for the transverse and longitudinal influence areas of the transmitted pressure are not comparable for all the units because of pavement temperature variation during the study.

Rigid Pavement Study

The rate of decrease of edge strain and corner deflection with increase in vehicle speed for the military vehicles was of the same order of magnitude as that observed for the conventional units. The values of strain and deflection at several axle loads for the military equipment were slightly lower than the values for the conventional units at the same wheel loads. Some of this difference was the result of pavement temperature differential.

The compressive edge strains recorded for the GOER and HETAG were found to be slightly lower than those for equivalent axle loads on conventional vehicles at all levels of speed and load included in this study. However, the corner deflection values for the HETAG were higher and for the GOER were lower than those recorded for the conventional units.

Dynamic Load Study

The heavy duty transporter (GOER) was equipped with instrumentation to record dynamic load effect. The findings from this study indicated a reversal of the trends found for other units. That is, an increase in dynamic load effect was normally associated with increase in vehicle speed and lower pavement serviceability. For the GOER, however, this relationship was found to exist for only one of the four conditions.

Bridge Study

The relationships of amplification factors to vehicle speed and vehicle class reported in Chapters 3, 4, and 5 were found to exist for this study as well.

The amplification factors for the HETAG and GOER were found to be appreciably higher than those for any of the conventional vehicles tested. Again, it was not possible to determine many important vehicle characteristics any of which might appreciably affect the findings.

A limited study of the dynamic effect of these vehicles on the strains in the deck slab indicated that the relationships of strain and vehicle speed and placement existed similar to those reported for the beam strains and deflections with the exception that the relationship between tandem and single axles was not shown.

6.2 SCOPE

Several units of military equipment were made available by the Department of Defense. The units included in this study were: M-52 tractor semitrailers (LPLS and conventional tires), heavy duty tank transporter (HETAG), double-ender tank transporter, two units of off-road train cargo trailers, 2 units of the rolling fluid transporter trailers and a self-propelled cargo-fluid transporter (GOER). Figure 53 shows these units as they were operated during the study.

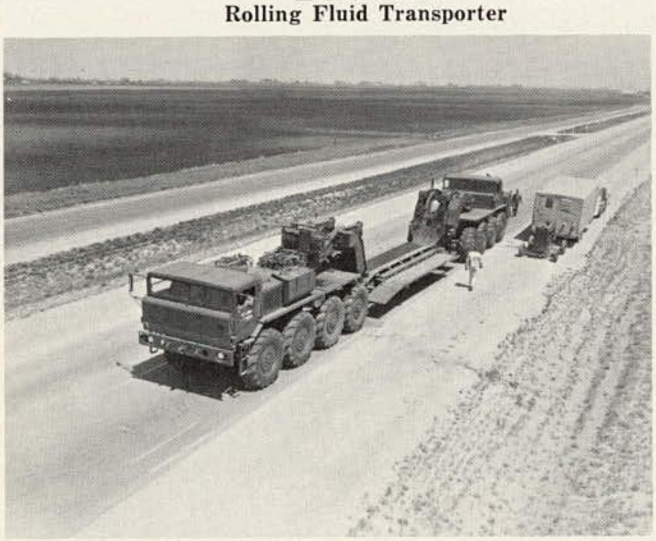
Table 54 gives the characteristics of the rolling fluid transporter, the off-road train and the M-52 tractor-semitrailers. A more comprehensive study was conducted making use of the GOER and HETAG. The characteristics of these units, including gross vehicle loading and individual axle loads, are given in Table

(K–1) HETAG
Tank Transporter

K–3 GOER
(Shown with dynamic load recording trailer)

K–5 and K–6
Rolling Fluid Transporter

K–7 and L–1
Off-Road Train Trailers



L–3
Heavy-Duty Tank Transporter

M–52
Tractor-Semitrailer

Figure 53. Military vehicles (tire) used in Special Study Program.

55. The numbers assigned to the units for the study are retained to facilitate the reporting of the data.

To evaluate the effect on pavements and bridges of the different units at different levels of speed and load, an investigation of strain, deflection, embankment pressure, and dynamic load was conducted on pavement sections in Loops 4 and 6 and on certain test bridges in Loop 6.

6.3 DESCRIPTION OF MEASUREMENTS

Table 27 lists the sections and structures selected for the study, their design and the in-

TABLE 54

VEHICLE CHARACTERISTICS

Vehicle	Tire Size	Tire Press. (psi)	Gross Vehicle Load (kips)	Axle Load (kips)					Gross Contact Area [1] (sq in.)	Gage (in.)	Axle Spacing (in.)
				1	2	3	4	5			
K-5[2]	64.00x45/18	5	2.5	2.5					435.0	58	—
K-6[2]	64.00x45/18	8	10.2	10.2					693.0	58	—
K-7[3]	64.00x48.5	16	44.2	21.7	22.5				331.0	110	133
L-1[3]	64.00x48.5	16	62.7	30.5	32.2				526.5	110	133
LPLS	46.00x24	35	71.2	9.6	15.1	15.1	15.3	16.1	204.1	72	—[4]
M-52	11.00x20	70	73.5	9.2	16.2	15.8	15.2	17.1	133.4	72	—[5]

[1] Area per tire based on measurements of limits of contact area.
[2] K-5, 6—Two 64x42, 18-ply tires mounted on a rigid axle with tow bar; filled with fluid to 8 psi (500 gal) each tire. For study, one unit filled with water (1,000 gal) at 8 psi and one unit empty at 5 psi. Units towed in tandem.
[3] K-7, L-1—Two units of off-road train cargo trailers towed individually by M-52 tractors; one unit loaded to approximately 20-kip gross load, other to 30-kip gross load.
[4] See Table 3.
[5] For characteristics of conventional units refer to Tables 26 and 38.

TABLE 55

VEHICLE CHARACTERISTICS HETAG AND GOER

Vehicle	Tire Size (in.)	Tire Press. (psi)	Vehicle Load	Axle Load (kips)								Axle Spacing (in.)						
				1[1]	2	3	4[2]	5[2]	6	7	8	1–2	2–3	3–4	4–5	5–6	6–7	7–8
K-1[3]	14.00x20/20	90	Empty	17.2	14.3	14.2	8.8	9.5	8.0	8.5	8.6							
			Empty	18.5	17.8	18.5	..	..	8.7	8.7	8.2							
			Blocks[4]		18.9	19.3	20.5	20.8	19.6	19.4	18.7							
			Blocks		32.7	33.9	..	..	19.0	18.8	18.4							
			K-4[5]		19.1	19.6	20.7	21.2	28.6	27.9	28.7							
			K-4		33.4	33.9	..	..	28.6	28.4	27.8							
			Blocks[6]		21.9	21.9	27.5	27.9	32.9	32.3	32.3							
			Blocks		42.5	43.3	..	..	32.2	31.6	31.8							
K-3[7]	29.5x25/16		Empty	23.4	13.4													
			Water[8]	30.2	25.8													
K-1	With dolly											192	60	69	58	406	58	58
	Without dolly											192	60	←	533	→	58	58
K-3												288						

[1] Steering axle.
[2] Axles on load transfer tandem dolly.
[3] K-1 = HETAG, heavy-duty tank transporter; Tractor semitrailer with removable load-divider tandem dolly; either 6 or 8 axles in combinations 1, 2–3, 6–7–8, or 1, 2–3, 4–5, 6–7–8.
[4] Approximately 20 kips of concrete blocks.
[5] M-47 tank loaded to rear of platform.
[6] Approximately 62.5 kips of concrete blocks.
[7] K-3 = GOER, a self-propelled cargo or fluid transporter resembling a conventional 2-axle tractor-scraper earthmover for use either on or off the highway sytem.
[8] Approximately 3,000 gal.

strumentation available in each. Figure 25 is a schematic drawing of typical pavement instrument installations, and Figure 35 is a schematic layout of the gages on the bridge structures. (See Chapter 3).

The vehicles were operated over the instrumented sections at several levels of speed, depending in some instances on the vehicle type and load. Some difficulty was experienced in scheduling the tests so as to minimize temperature effects because of late delivery of some of the special vehicles. However, control vehicles (conventional tractor semitrailers) were operated in conjunction with the military vehicles for each study. In the discussion that follows, the effect of temperature has been taken into consideration.

The data and observations are reported for the flexible pavement study, the rigid pavement study, the dynamic load study and the bridge study.

6.4 FLEXIBLE PAVEMENT STUDY

Relationships were developed between axle load and deflection, and between vehicle speed and deflection, and studies were made of the pressure transmitted to the embankment soil and of the dynamic load effect (GOER only). Where possible, comparisons are shown with similar relationships for conventional vehicles.

Table 56 gives the outer wheel path total deflections for sections 265 and 333 for all the units except the HETAG and the GOER. Deflections under conventional vehicles at several axle loads are also given. The two axle loads on each of the vehicles, K-7 and L-1, vary by only about 1 kip (see Table 54). Thus, for this study the mean deflections for the two axles were used.

M-52 tractors were used as towing units for the off-road train and for the rolling fluid transporter trailers. The maximum safe speed for these units as they passed the instrument

TABLE 56

FLEXIBLE PAVEMENT DEFLECTIONS, OUTER WHEELPATH

Vehicle	Axle Load (kips)	Deflection (10^{-3} in.) Section 265 Creep	Section 265 15 Mph	Section 265 30 Mph	Section 333 Creep	Section 333 15 Mph	Section 333 30 Mph
Conv.	12 S	9	7	7	14	12	10
Conv.	18 S	19	14	14	25	20	17
Conv.	24 T	15	12	12	20	16	14
Conv.	30 S	20	17	15	33	26	23
Conv.	32 T	16	14	12	21	18	15
M	32 T	15	13	14	20	15	16
G (LPLS)	32 T	17	16	14	25	20	20
K–5	2.5 S	3	2	—	3	2	—
K–6	10.2 S	10	8	—	10	9	—
K–7	22.1[1] S	19	17	—	26	23	—
L–1	31.3[2] S	21	20	—	34	34	—

[1] Mean axle load: Axle 1, 21.7 kips; Axle 2, 22.5 kips.
[2] Mean axle load: Axle 1, 30.5 kips; Axle 2, 32.2 kips.

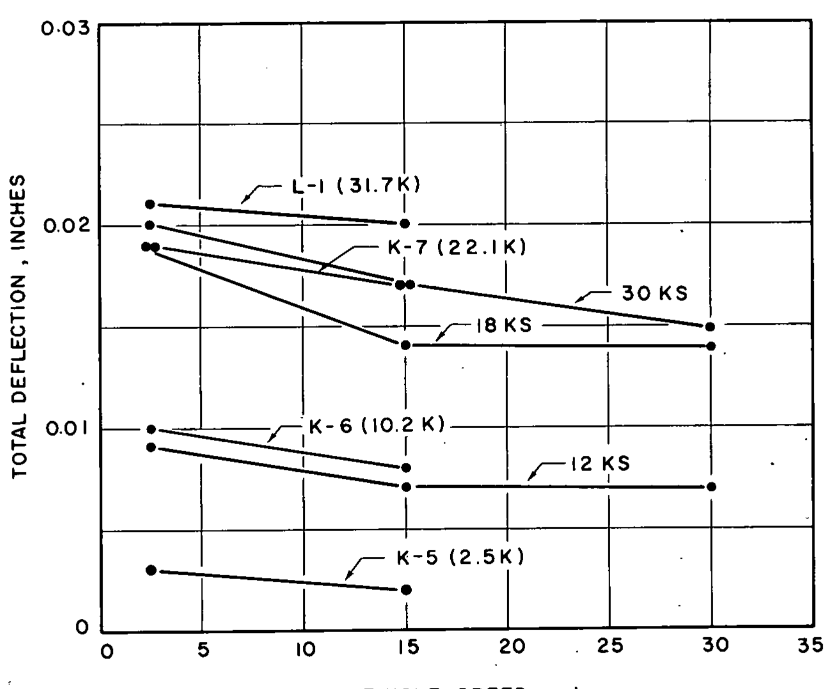


Figure 54. Relationship between vehicle speed and deflection, Section 265 (5-9-16 design).

TABLE 57

FLEXIBLE PAVEMENT DEFLECTIONS

Vehicle	Net Load	Load Divider Dolly	Axle Load (kips)	Deflection (10^{-3} in.)					
				Section 265 (5–9–16)			Section 333 (6–9–16)		
				Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph
HETAG	Empty	Yes	14.3[1]	13	10	9	18	12	12
			14.2	13	10	11	19	13	13
			8.8	9	7	7	14	8	9
			9.5	9	7	7	14	9	10
			8.0	7	5	5	10	7	7
			8.5	8	6	6	10	7	9
			8.6	7	5	5	9	7	8
HETAG	20 kips	Yes	18.9	17	13	13	20	17	13
			19.3	17	13	14	21	18	14
			20.5	17	14	15	21	18	16
			20.8	18	14	16	23	19	19
			19.6	16	12	12	18	16	11
			19.4	18	13	13	19	16	13
			18.7	18	12	13	18	16	15
HETAG	20 kips	No	32.7[2]	27	23	22	36	31	29
			33.9	27	24	24	38	31	29
			19.0	16	14	12	22	19	17
			18.8	17	14	13	24	19	18
			18.4	16	15	14	24	19	17
HETAG	K–4	Yes	19.1	19	14	13	29	21	17
			19.6	21	14	15	30	21	19
			20.7	21	16	15	32	24	20
			21.2	22	16	16	33	25	22
			28.6	23	19	17	33	28	25
			27.9	25	21	19	36	30	27
			28.7	26	21	20	37	31	27
HETAG	K–4	No	33.4	27	22	20	35	28	22
			33.9	29	23	22	37	29	26
			28.6	25	18	17	29	23	20
			28.4	27	21	19	31	24	22
			27.8	27	20	20	32	25	22
HETAG	62.5 kips	Yes	21.9	17	15	13	24	21	18
			21.9	19	16	15	28	23	20
			27.5	23	19	18	33	28	24
			27.9	24	21	19	37	30	26
			32.9	24	21	19	38	31	26
			32.3	25	22	21	41	33	31
			32.3	26	22	22	42	35	32
HETAG	62.5 kips	No	42.5	29	28	25	45	25	32
			43.3	30	28	27	49	39	36
			32.2	25	24	19	37	30	27
			31.6	27	25	22	40	35	31
			31.8	27	25	22	41	35	35
GOER	Empty	—	23.4[3]	16	15	13	27	25	22
			13.4	12	11	10	18	18	15
GOER	3,000 gal	—	30.2[4]	22	21	21	41	39	32
			25.8	21	20	19	35	35	31

[1] Steering axle not included; axles 2, 3, 4, 5, 6, 7, 8.
[2] Steering axle not included; axles 2, 3, 6, 7, 8.
[3] Tire inflation pressure 25 and 20 psi.
[4] Tire inflation pressure 35 and 30 psi.

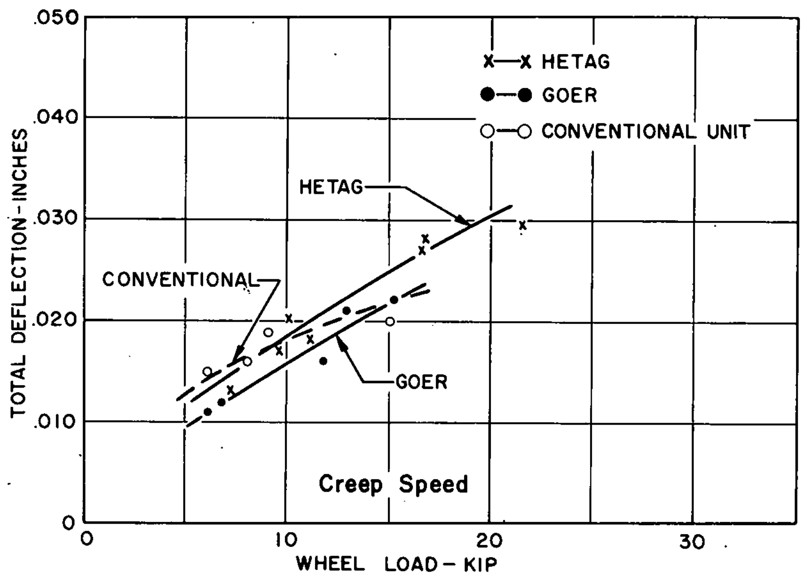


Figure 55. Relationship between deflection and wheel load, Section 265 (5-9-16 design).

vans was 15 mph; therefore, no deflection data for 30 mph are available.

The relationships between vehicle speed and deflection for these vehicles compare favorably with the relationships reported for other vehicles. That is, the pavement deflection decreases as the speed increases. The relationships between vehicle speed and deflection for conventional and military units at or near the same axle loads are shown in Figure 54. The slope of the relationships and the relative position of each unit or axle load on the deflection scale agrees with the relationship between the load and deflection previously reported.

The same relationships shown for section 265 can be developed for section 333. The magnitude of the deflection is the only noticeable difference between the relationships for the two sections. Section 333 had a lower serviceability than section 265 and its deflections were correspondingly higher.

The deflection data for the GOER and HETAG are given in Table 57. Several net loads (vehicles and concrete blocks) were placed on both these units, and the axle loads resulting from these are noted. The relationships between vehicle speed and deflection for both of these units at all axle loads are similar to those reported for the other military vehicles.

With the data for several axle loads available, the wheel load and deflection relationship can be shown. A power function relationship was assumed to exist between wheel load and deflection (Report 5).

Figure 55 shows this relationship for the HETAG and the GOER at creep speed. The deflections for axles 2 and 3 on the HETAG were used in developing these relationships. All axle loads for the GOER were used. The curves for both units show a greater rate of increase of deflection with increase in axle load than for the conventional vehicles. For equal axle loads, the deflection values for the HETAG were appreciably greater than those for the GOER.

The maximum values of pressure transmitted to the embankment soil for the military vehicles (excluding the HETAG and GOER) are given in Table 58. Pressure values for a range of axle loads on conventional vehicles are

TABLE 58

PRESSURE TRANSMITTED TO EMBANKMENT SOIL, SECTION 581 (5-6-12 DESIGN)

Vehicle	Axle Load (kips)	Zero Reading (ft)		Maximum Pressure (psi)		Zero Reading (ft)	
		Creep	15 Mph	Creep	15 Mph	Creep	15 Mph
Conv.	12 S	3.3	4.5	4.9	4.5	5.0	4.5
Conv.	18 S	4.5	4.4	6.1	5.4	5.1	5.4
Conv.	32 T	4.3	4.5	5.2[1]	4.5	5.0	5.4
Conv.	30 S	4.5	4.6	10.2	8.4	4.9	5.2
K-5	2.5 S	3.0	4.0	0.6	0.5	3.0	5.0
K-6	10.2 S	4.3	5.0	2.5	2.2	4.7	6.0
K-7	22.1 S	5.3	5.5	6.1[1]	4.5	5.8	6.5
L-1	31.3 S	5.3	6.0	7.3[1]	5.9	5.7	6.6

[1] Mean value of both loaded trailer axles.

TABLE 59

PRESSURE TRANSMITTED TO EMBANKMENT SOIL, SECTION 581 (5–6–12 DESIGN)

Axle Load (kips)	Maximum Pressure (psi)		
	Creep	15 Mph	30 Mph
	(a) HETAG		
14.3	4.4	3.7	..
14.2	4.4	3.7	..
8.8	3.1	2.6	..
9.5	3.2	2.6	..
8.0	2.8	2.0	..
8.5	2.9	2.3	..
8.6	3.0	2.8	..
18.9	5.5	4.1	..
19.3	5.2	4.4	..
20.5	5.8	4.7	..
20.8	6.0	4.7	..
19.6	5.0	4.4	..
19.4	5.3	4.5	..
18.7	5.3	4.4	..
32.7	8.2	7.1	..
33.9	8.5	7.5	..
19.0	4.7	4.1	..
18.8	5.0	4.3	..
18.4	4.8	4.3	..
19.1	5.0	5.5	4.6
19.6	5.2	5.0	4.6
20.7	5.8	5.3	4.8
21.2	6.2	5.9	5.2
28.6	7.1	7.0	6.2
27.9	7.7	7.4	6.6
28.7	7.8	7.4	6.6
33.4	8.0	6.2	5.9
33.9	7.8	6.8	6.6
28.6	6.2	5.3	4.6
28.4	6.6	5.9	5.2
27.8	6.4	5.7	5.0
21.9	6.4	6.2	5.5
21.9	6.2	6.4	5.0
27.5	8.4	7.8	6.4
27.9	8.6	8.3	7.3
32.9	8.9	9.1	..
32.3	9.4	9.1	..
32.3	9.6	9.3	..
42.5	12.1	11.0	10.7
43.3	12.8	12.5	10.8
32.2	9.6	8.9	..
31.6	9.8	9.1	..
31.8	9.8	8.9	..
	(b) GOER		
23.4	9.5	7.7	..
13.4	5.9	5.0	..
30.2	13.0	11.8	..
25.8	9.1	10.5	..

also given. The pressure transmitted to the embankment soil decreased with an increase in vehicle speed for all vehicles. In general, the data also show that an increase in wheel load (one-half axle load) caused an increase in pressure transmitted to the embankment soil. However, because of the wide variety of tire sizes and designs on this equipment, no graphic presentation was attempted.

The effect of vehicle speed on the distance from the loaded axle (leading and trailing) at which a zero embankment pressure was recorded was the same as that experienced for other vehicles. That is, an increase in distance was associated with an increase in vehicle speed. A slight increase in the distance was also associated with an increase in axle load.

The pressures transmitted to the embankment soil under the several loads on the HETAG and GOER are given in Table 59. An appreciable decrease of embankment pressure with an increase in vehicle speed was apparent for these units at all axle loads. The wheel load and embankment pressure relationship is shown in Figure 56. The values of transmitted pressure for the GOER were appreciably higher than for the HETAG, and the increase of pressure transmitted to the embankment soil with increase in wheel load was greater at the higher wheel loads for both units.

The transverse distribution of the embankment pressure at creep speed is shown in Figure 57. These relationships were developed from the data given in Table 60. The diagrams for pressure transmitted to the embankment soil for a speed of 15 mph were similar to those at creep speed except for lower maximums and greater length of influence of pressure.

The slope and position of the curves for the military units compared favorably with those for the conventional units at equal wheel loads, except for L-1 (31.3 kip axle load) and the conventional 30-kip single axle vehicles. This difference can be explained, at least partially, by the fact that the pavement temperature was 20 deg lower when the tests on vehicle L-1 were conducted.

6.5 RIGID PAVEMENT STUDY

Table 61 shows deflection and strain data for the military equipment (except the HETAG and GOER) for sections 389 and 397, Loop 6. The values of tensile and compressive strain and deflection decreased with an increase in vehicle speed at about the same rate for all military and conventional units.

In general, the values of compressive strain and deflection for the military units were lower than the values for conventional units at or near the same axle load. This was true for both sections and for vehicle speeds of creep

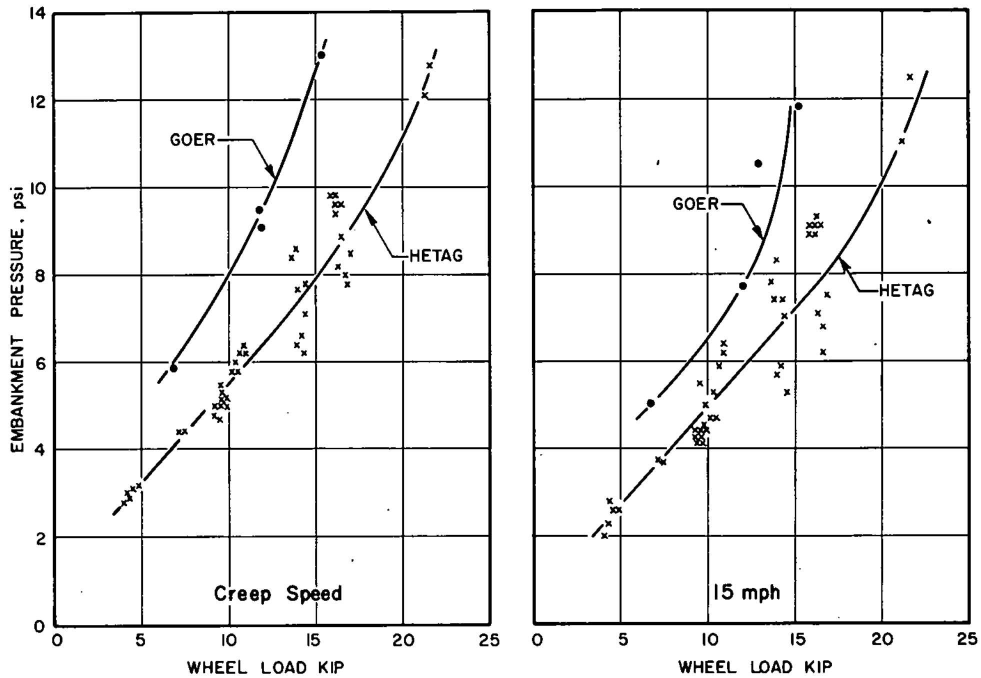


Figure 56. Relationship between wheel load and embankment pressure, Section 581 (5-6-12 design).

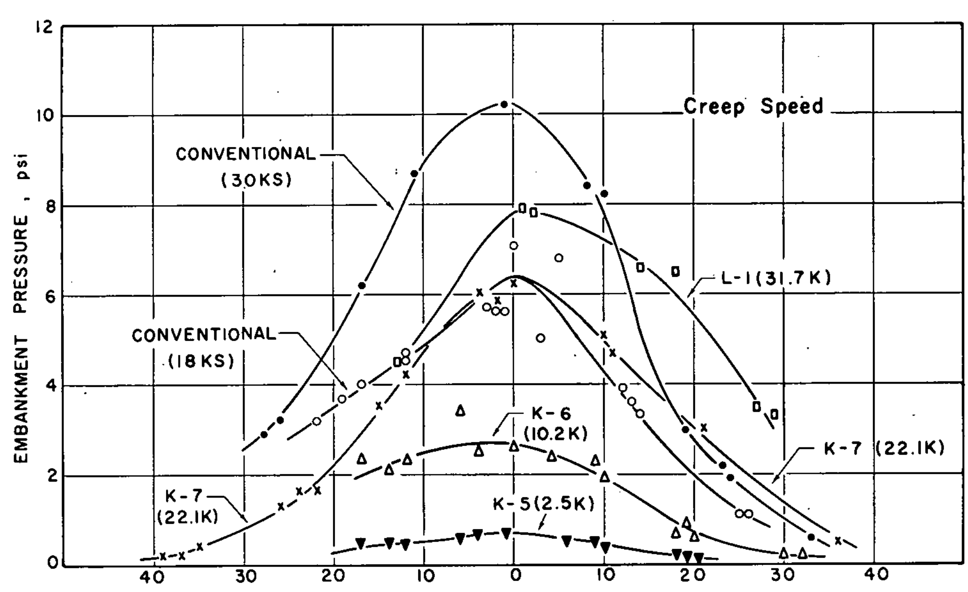


Figure 57. Transverse profile, embankment pressure, Section 581 (5-6-12 design).

TABLE 60

EMBANKMENT PRESSURE TRANSVERSE DISTRIBUTION, SECTION 581 (DESIGN 5–6–12)

Vehicle and Load	Creep Speed		15 Mph	
	Distance from Gage (in.)	Press. (psi)	Distance from Gage (in.)	Press. (psi)
Conv. (18KS)	22	3.2	11	3.7
	19	3.7	8	4.3
	17	4.0	7	3.4
	12	4.7	2	6.1
	12	4.6	1	4.8
	3	5.7	0	5.4
	2	5.6	2	4.6
	1	5.6	4	4.1
	0	7.1	9	3.4
	3	5.0	16	2.0
	5	6.8	29	0.6
	12	3.9		
	13	3.6		
	14	3.3		
	25	1.1		
	26	1.1		
K–6 (10.2K)	17	2.3	3	2.0
	14	2.1	2	2.1
	12	2.3	0	2.2
	6	3.4	5	1.8
	4	2.5	8	1.8
	1	2.6	18	1.8
	0			
	6	2.4	20	0.8
	9	2.3	33	0.2
	10	1.9	35	0.1
	18	0.7		
	19	0.9		
	20	0.6		
	30	0.2		
	32	0.2		
Conv. (30KS)	28	2.9	25	3.2
	26	3.2	23	3.4
	17	6.2	15	7.1
	11	8.7	12	7.8
	1	10.2	7	8.4
	0			
	8	8.4		
	10	8.2		
	19	3.0		
	22	2.2		
	33	0.6		
K–5 (2.5K)	17	0.5	3	0.6
	14	0.5	2	0.5
	12	0.5	0	0.5
	6	0.6	5	0.5
	4	0.7	8	0.5
	1	0.7	18	0.5
	0			
	6	0.5	20	0.1
	9	0.5		
	10	0.4		
	18	0.2		
	19	0.2		
	20	0.1		
	30	0.1		
K–7 (22.1K)	39	0.2	24	1.9
	37	0.2	23	1.6
	35	0.4	22	1.9
	26	1.3	11	3.3
	24	1.6	11	3.6
	22	1.6	0	4.5
	15	3.5	8	4.5
	12	4.2	12	3.6
	4	6.0	24	2.0
	2	5.8	25	1.8
	0	6.2	34	0.8
	10	5.1	35	0.6
	11	4.7		
	21	2.5		
	24	1.9		
	36	0.5		
Conv. (32KT)	26	1.6	10	3.4
	25	1.9	9	3.6
	22	2.5	2	3.9
	14	3.8	1	4.8
	13	4.0	0	4.8
	3	4.5	1	4.7
	2	5.1	2	3.9
	1	5.3	4	3.8
	0	4.9	5	3.6
	1	5.2	14	2.0
	2	5.2	15	1.7
	3	4.8	23	0.8
	4	4.7	24	0.8
	8	3.5		
	10	3.4		
	18	1.0		
	26	0.6		
L–1 (31.3K)	13	4.3	21	1.9
	0	—	20	2.3
	1	7.9	11	4.5
	2	7.8	8	4.5
	3	7.7	1	5.9
	14	6.6	0	6.1
	18	6.5	13	5.8
	27	3.5	15	6.2
	29	3.3	27	3.2
	32	1.3	36	0.9

and 15 mph. Some of this difference can be attributed to the influence of the temperature differential in the pavement surface when the different tests were run. Vehicle breakdowns and instrumentation delays made it necessary to continue the studies with these units over different days with different temperatures in the pavement surface.

Table 62 lists the values of compressive and tensile strain and deflection for the GOER and the HETAG. With each loading for the HETAG, the maximum compressive strain per axle is given, but the maximum tensile strain and deflection values are given for only the heaviest axle load. This is true for the tensile strain values for the GOER as well. The relationships reported between vehicle speed and strain or deflection for all other units existed for these vehicles as well.

The relationship between axle load and compressive strain, and axle load and deflection, at creep speed for these two units is shown in Figure 58. The comparable relationships for the conventional vehicles are also shown.

6.6 DYNAMIC LOAD STUDY

The equipment and test procedure described in Chapter 3 (Section 3.6) was adapted for use with the GOER only. Major modifications would have been required to make dynamic load studies with the other military vehicles. Figure 59 shows the GOER with the instrument trailer for recording dynamic load effect.

Table 63 gives the data taken while operating the GOER over the two pavement sections in Loop 6 (Chapter 3, Section 3.6). Findings previously reported in Chapters 3, 4 and 5 indicate that one would expect an increase in dynamic load effect with increase in speed and a decrease in pavement serviceability. There was no such consistent trend for these data. For the empty load and the rough pavement condition the relationship was as expected; but for the other three load and pavement conditions reversals were indicated.

The dynamic load effect when the vehicle was subject to impact forces is reported in Chapter 8.

TABLE 61

OBSERVATIONS, RIGID PAVEMENT

Vehicle	Axle Load	Strain (10^{-6} in./in.)						Corner Deflection (10^{-3} in.)		
		Compressive			Tensile					
		Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph
(a) Section 389, 9.5–6 Design										
Conv.	12 S	18	12	11	7	6	4	15	11	9
Conv.	18 S	23	19	18	12	11	10	23	21	19
Conv.	24 T	15	13	12	13	12	11	29	26	25
Conv.	30 S	32	28	25	16	15	13	45	38	32
Conv.	32 T	20	17	14	13	13	10	34	32	29
M–52	32 T	16	18	16	13	11	10	21	18	17
LPLS	32 T	17	18	17	12	12	12	20	19	19
K–5	2.5	1	—	—	4	—	—	3	4	—
K–6	10.2	8	8	—	4	4	—	8	7	—
K–7	22.1	18	18	—	7	5	—	12	12	—
L–1	31.3	24	19	—	10	6	—	19	15	—
(b) Section 397, 11–6 Design										
Conv.	12 S	11	10	8	4	4	5	9	8	8
Conv.	18 S	19	17	15	9	8	8	17	14	13
Conv.	24 T	14	12	12	9	8	8	19	16	16
Conv.	30 S	26	24	21	14	12	11	30	24	20
Conv.	32 T	17	15	14	12	10	8	24	21	19
M–52	32 T	17	15	—	12	11	—	18	15	—
LPLS	32 T	16	15	—	12	11	—	18	16	—
K–5	2.5	7	7	—	3	3	—	5	6	—
K–6	10.2	4	4	—	3	3	—	4	4	—
K–7	22.1	14	14	—	6	5	—	10	8	—
L–1	31.3	17	16	—	10	8	—	19	15	—

TABLE 62

OBSERVATIONS, RIGID PAVEMENT

Vehicle	Net Load	Load Divider Dolly	Axle Load (kips)	Mean Compressive Strain (10^{-6} in./in.)					
				Section 389 (9.5–6)			Section 397 (11–6)		
				Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph
HETAG	Empty	Yes	14.3	10	10	8	6	6	4
			14.2	9	9	10	7	6	6
			8.8	4	5	3	2	2	2
			9.5	6	7	6	5	4	4
			8.0	6	6	5	4	3	2
			8.5	8	8	8	5	5	4
			8.6	7	7	7	5	5	4
Max. mean tens. (10^{-6} in./in.)				*10*	*9*	*11*	*13*	*11*	*9*
Max. mean defl. (10^{-3} in.)				*19*	*16*	*16*	*25*	*21*	*16*
HETAG	K–4	Yes	19.1	12	12	9	7	7	7
			19.6	10	10	8	7	5	6
			20.7	13	12	10	9	8	8
			21.2	18	18	15	13	12	12
			28.6	21	20	17	13	12	12
			27.9	24	24	20	18	17	17
			28.7	24	24	20	17	17	16
Max. mean tens. (10^{-6} in./in.)				*13*	*13*	*12*	*13*	*13*	*12*
Max. mean defl. (10^{-3} in.)				*40*	*36*	*34*	*35*	*30*	*28*
HETAG	K–4	No	33.4	26	28	22	22	20	19
			33.9	31	32	26	28	23	21
			28.6	16	17	14	14	9	12
			28.4	20	19	16	18	13	14
			27.8	19	19	16	17	13	14
Max. mean tens. (10^{-6} in./in.)				*25*	*23*	*20*	*20*	*18*	*18*
Max. mean defl. (10^{-3} in.)				*55*	*47*	*40*	*44*	*38*	*40*
HETAG	20K	Yes	18.9	12	13	11	8	8	7
			19.3	12	10	10	8	8	7
			20.5	13	12	11	10	11	9
			20.8	18	15	16	14	15	12
			19.6	13	13	12	10	10	8
			19.4	14	15	15	13	13	11
			18.7	14	15	14	13	13	11
Max. mean tens. (10^{-6} in./in.)				*11*	*10*	*9*	*8*	*8*	*9*
Max. mean defl. (10^{-3} in.)				*27*	*25*	*23*	*23*	*22*	*20*
HETAG	20K	No	32.7	25	22	22	21	18	18
			33.9	30	27	24	27	24	23
			19.0	13	9	8	9	8	6
			18.8	13	10	7	12	22	9
			18.4	13	12	8	12	11	9
Max. mean tens. (10^{-6} in./in.)				*26*	*23*	*21*	*25*	*23*	*21*
Max. mean defl. (10^{-3} in.)				*62*	*53*	*54*	*49*	*43*	*39*
HETAG	6.25K	No	42.5	33	27	25	29	25	24
			43.3	40	32	30	38	32	32
			32.2	20	17	16	17	14	13
			31.6	23	19	17	24	18	18
			31.8	23	19	18	23	19	18
Max. mean tens. (10^{-6} in./in.)				*29*	*21*	*18*	*25*	*23*	*21*
Max. mean defl. (10^{-3} in.)				*71*	*65*	*65*	*46*	*39*	*35*
GOER	Empty	—	23.4	25	21	20	20	18	17
Max. mean tens. (10^{-6} in./in.)				*7*	*6*	*6*	*5*	*5*	*4*
Max. mean defl. (10^{-3} in.)				*21*	*18*	*18*	*15*	*14*	*12*
			13.4	15	12	12	11	10	9
Max. mean defl. (10^{-3} in.)				*14*	*11*	*11*	*9*	*8*	*8*
GOER	3,000 gal	—	30.2	30	29	27	26	29	27
Max. mean tens. (10^{-6} in./in.)				*13*	*11*	*10*	*11*	*10*	*10*
Max. mean defl. (10^{-3} in.)				*43*	*38*	*32*	*29*	*24*	*22*
			25.8	25	20	21	20	22	19
Max. mean defl. (10^{-3} in.)				*36*	*33*	*28*	*26*	*22*	*20*

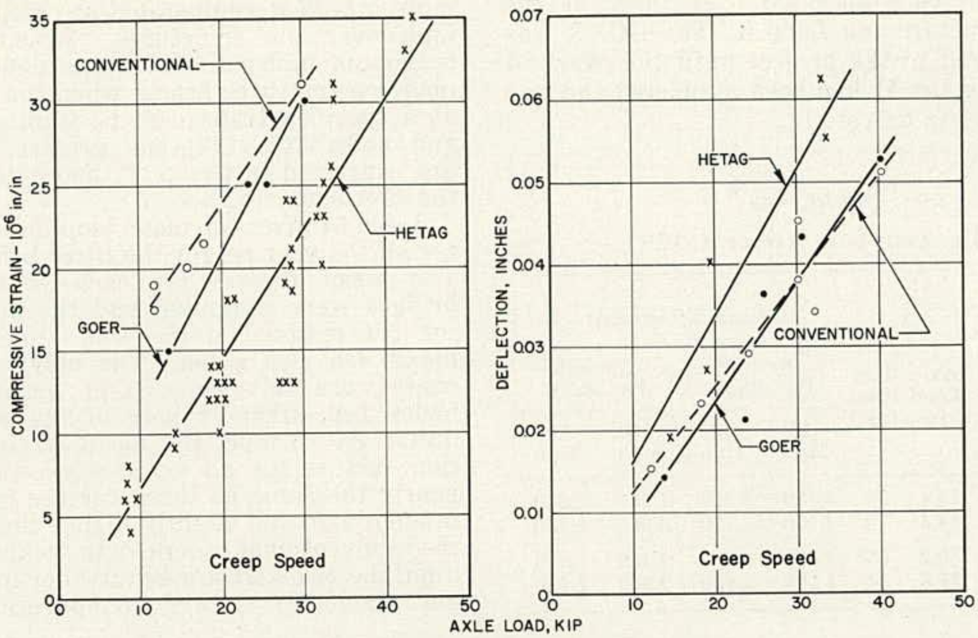


Figure 58. Relationship between axle load and compressive strain and deflection, Section 389 (9.5-6 design).

Figure 59. Calibration of dynamic load-tire pressure equipment on GOER.

6.7 BRIDGE STUDY

Each of the military vehicles listed in Tables 54 and 55 were operated over three of the bridge structures on Loop 6. The GOER was not delivered to the project until the overload studies (Report 4) had been completed and was not included in this study.

TABLE 63

DYNAMIC LOAD EFFECT, GOER

Net Load	Axle Load (kips)	Tire Pres. (psi)	Load Effect (lb): Rough Pavement 10 Mph	Rough Pavement 20 Mph	Smooth Pavement 10 Mph	Smooth Pavement 20 Mph
Empty	23.4	25	4,130	8,820	4,550	3.030
	13.4	20	4,070	7,770	3,050	2,150
3,000 gal	30.2	35	4,890	4,080	4,120	3,570
	25.8	30	4,190	2,680	2,360	2,120

The study was conducted similarly to that described in Chapter 3, Section 3.7. Some of the vehicles were operated at creep and 15 mph; others were operated at creep, 15 and 30 mph over the structures. Measurements of maximum midspan strain and deflection were made on the three beams when the vehicle was at a specified transverse position. The strain and deflection at speeds greater than creep are expressed in terms of their relationship to the effects at creep speed.

Table 64 gives the mean amplification factors for all the vehicles for the three bridges tested. The mean factors for each vehicle for all bridges were computed and the mean factors for conventional single and tandem axle vehicles are also given. The only comparisons made were between each of the military vehicles and either or both of the conventional units. At 15 mph, the mean strain amplification factors for all vehicles except K-6 were nearly the same as those for the conventional tandem axle and slightly higher than those for the conventional single-axle vehicle. At 30 mph, the M-52 tractor-semitrailer units (LPLS and standard) showed an appreciably higher

TABLE 64

MEAN AMPLIFICATION FACTORS, BRIDGES 3B, 8A AND 8B, LOOP 6

Vehicle	Axle Load (kips)	Factor for Bridge Indicated: 3B 15 Mph	3B 30 Mph	8A 15 Mph	8A 30 Mph	8B 15 Mph	8B 30 Mph	Mean 15 Mph	Mean 30 Mph
		(a) STRAIN AMPLIFICATION FACTOR							
K-1	—[1]	1.007	1.153	1.029	1.197	1.117	1.200	1.051	1.183
	—[2]	1.007		1.029		1.013		1.016	
K-6	10.2	1.272	—	1.142	—	1.112		1.175	
K-7	21.7 and 22.5	1.115	—	1.064	—	1.022		1.067	
L-1	30.5 and 32.2	1.038		1.078		1.115		1.077	
Conv.	18 S	1.140	1.100	1.149	1.195	1.072	1.168	1.120	1.154
Conv.	32 T	1.027	1.027	1.099	1.015	1.081	1.032	1.069	1.024
LPLS	32 T	1.023	1.154	1.027	1.081	1.037	1.187	1.029	1.140
M-52[3]	32 T	1.060	1.216	1.078	1.108	1.052	1.156	1.063	1.160
		(b) DEFLECTION AMPLIFICATION FACTOR							
K-1	—[1]	1.010	1.139	1.029	1.171	1.038	1.100	1.025	1.136
	—[2]	1.034		1.007		1.115		1.052	
K-6	10.2	1.269		1.130		1.125		1.174	
K-7	21.7 and 22.5	1.102		1.100		1.072		1.091	
L-1	30.5 and 32.2	1.086		1.101		1.150		1.112	
Conv.	18 S	1.050	1.200	1.100	1.216	1.150	1.226	1.100	1.214
Conv.	32 T	1.017	1.125	1.152	1.094	1.093	1.080	1.087	1.099
LPLS	32 T	1.000	1.166	1.041	1.061	1.058	1.174	1.033	1.133
M-52[3]	32 T	1.015	1.215	1.051	1.113	1.069	1.139	1.045	1.155

[1] 18.9, 19.3, 20.5, 20.8, 19.6, 19.4, 18.7 kips for axles 2, 3, 4, 5, 6, 7, 8, respectively.
[2] 32.7, 33.9, 19.0, 18.8, 18.4 kips for axles 2, 3, 6, 7, 8 respectively.
[3] Conventional military tread tires, 70 psi.

TABLE 65

STRAIN AMPLIFICATION FACTORS [1]

Vehicle	Axle Load (kips)	Amplification Factor							
		15 Mph				30 Mph			
		Gage 1	Gage 2	Gage 3	Mean	Gage 1	Gage 2	Gage 3	Mean
(a) BRIDGE 3B									
K-1	— [2]	0.986	0.960	1.000	0.993	0.995	1.214	1.262	1.165
K-6	10.2 S	0.942	1.016	1.104	1.030				
K-7	— [3]	0.923	1.016	0.833	0.921				
L-1	— [4]	0.934	1.021	0.969	0.974				
H-3	— [5]	1.077	1.023	1.015	1.043				
H-6	— [6]	0.934	1.016	0.915	0.958				
M-52	32 T	1.023	1.033	1.017	1.024				
LPLS	32 T	1.039	1.014	1.102	1.055				
Conv.	32 T	1.107	1.136	1.094	1.116	0.966	1.025	0.993	0.994
Conv.	40 T	1.038	1.042	1.048	1.039				
Conv.	18 S	1.054	1.037	1.152	1.079				
Mean		1.005	1.029	1.022					
(b) BRIDGE 8A									
K-1	— [2]	1.027	1.048	1.047	1.040	1.076	1.125	1.134	1.113
K-6	10.2 S	1.171	1.118	1.065	1.125				
K-7	— [3]	0.888	0.967	0.874	0.908				
L-1	— [4]	1.108	0.859	1.039	1.005				
H-3	— [5]	0.940	1.014	0.916	0.957				
H-6	— [6]	0.779	0.970	0.749	0.832				
M-52	32 T	0.966	1.131	1.055	1.047				
LPLS	32 T	1.086	1.067	1.020	1.058				
Conv.	32 T	1.096	1.086	1.149	1.114	1.303	1.052	1.149	1.180
Conv.	40 T	1.015	1.048	1.031	1.025				
Conv.	18 S	1.042	1.107	1.033	1.058				
Mean		1.010	1.037	0.998					

[1] Deck slab strain gages located at one-third points in direction of traffic.
[2] 18.9, 19.3, 20.5, 20.8, 19.6, 19.4, 18.7 kips on axles 2, 3, 4, 5, 6, 7, 8, respectively.
[3] 21.7, 22.5 kips on axles 1, 2, respectively.
[4] 30.5, 32.2 kips on axles 1, 2, respectively.
[5] Small scraper unit axle loads of 47.3 and 40.7 kips.
[6] Medium scraper unit, axle loads of 61.2 and 48.8 kips.

strain amplification factor than the conventional tandem axle units. This was also true for vehicle K-1.

Similar relationships existed for the mean deflection amplification factors at both speeds. Vehicle K-6 showed a higher factor than the conventional tandem unit; all other units showed an equal or lower factor.

The effects of vehicle speed and class of vehicle on the amplification factors were similar to those discussed in Chapter 3.

These relationships cannot be considered to be more than general trends. The variations among the military vehicles in the test made direct comparisons of little value. A thorough analysis of the vehicle and bridge characteristics involving parameters that were not measured would be required before meaningful relationships might be established.

Strain gages were installed on the transverse tension reinforcing steel in the reinforced concrete deck slabs on bridges 3B and 8A to investigate the dynamic effect of a variety of vehicles on the bridge deck slabs. The gages were installed at the longitudinal one-third points and between the exterior and center bridge beams. Maximum strains were measured at speeds of creep and 15 mph, and some data were taken at 30 mph when the outer loaded wheels of the vehicles passed directly over the gage point.

The strain amplification factors for each of the vehicles and bridges tested are given in Table 65. The mean factors for the three gage points were computed for each vehicle and for all the vehicles. The relationship between the amplification factors for tandem and single axle vehicles (reported in Chapters 3, 4 and 5) did not exist in this study. That is, the amplification factor for the tandem axle units was not consistently lower than the factors for the single axle units.

In the speed study, the increase in amplification factor with increase in speed was not consistent for the vehicles tested.

The mean amplification factors for each of the three gage points indicated that the dynamic strains at gage 2 were slightly higher than the strains at gages 1 and 3.

6.8 NEEDED RESEARCH

The time limitations for this study made it difficult to compare the dynamic effects on pavements and bridges of military and conventional units.

Future investigations should be designed so that detailed studies at several axle loads, vehicle speeds and transverse placements might be conducted. Instrumentation for detecting strains, deflections and transmitted pressures should be located at points on or about the pavement structure different from those on the Road Test.

The dynamic load effect records for the GOER should be reviewed, assuming a different summary method. The results reported in this study indicated that the normal trend of this effect with vehicle speed and pavement serviceability did not exist.

Further modifications of the dynamic load-tire pressures instrumentation might make it adaptable to some of the other axle configurations.

For the heavy duty transporters, designed for either off or on highway usage, a performance study comparing these units with conventional vehicles would be desirable.

Chapter 7

Military Vehicles, Track

7.1 SUMMARY

The objective of this study was to investigate the dynamic effect on pavements of track-laying military equipment and to compare these effects to those of conventional equipment.

It was evident after conducting this study and reviewing the data that the instrumentation available at the Road Test was not adaptable to the track-laying equipment insofar as the determination of dynamic effects on the pavements and bridges was concerned.

In general, the relationships between vehicle speed and deflection (rigid and flexible) and between vehicle speed and edge compression strains for the track equipment indicated the same trend, as for conventional units.

7.2 SCOPE

An M-47 tank and a T-113 personnel carrier (Fig. 60) were made available to the project by the Department of Defense for this portion of the study.

To protect the electronic instrumentation within the pavement, the studies on these units were delayed until completion of all the previously described special studies. At this time overload studies on the bridge structures had been completed, thus no bridges were available for study under the track-laying units.

The characteristics of these track-laying vehicles are given in Table 66.

7.3 DESCRIPTION OF MEASUREMENTS

A limited investigation of dynamic strains and deflections and transmitted pressures was undertaken on one flexible section (265) and one rigid section (389) in Loop 6 and one flexible section (581) on Loop 4. (See Table 27.)

A train of vehicles, including conventional units with 12-kip, 18-kip and 30-kip single and 32-kip tandem axle loads at creep speed, 15 and 30 mph and the tracked vehicles at creep speed and 15 mph, was operated over the instrumented sections.

Figure 60. Military vehicle (tracked), T-113 personnel carrier.

7.3.1 Flexible Pavements

Table 67 gives the inner wheelpath deflections (total, embankment and subbase) under the vehicles in the study for section 265. Each of the values is a mean of a minimum of four recorded values.

Figure 61 shows the relationship of vehicle speed to total deflection of the flexible pavement. The shapes of the curves and the magnitude of the values for the conventional vehicles are similar to those found in the other studies reported. Deflection was a function of speed for the conventional and the light tracked vehicle. No reduction in deflection with speed was noted for the heavy tracked vehicle over the range of speed studied.

The values of embankment pressure in section 581 under these vehicles are given in Table 68. The vehicles were operated directly over the gage point at speeds of creep and 15 mph. The transverse pressure distribution at both levels of speed is shown in Figures 62 and 63.

The curves for the conventional and military tracked vehicles are uniform and have the same relationship at both vehicle speeds. It was interesting that the embankment pressure

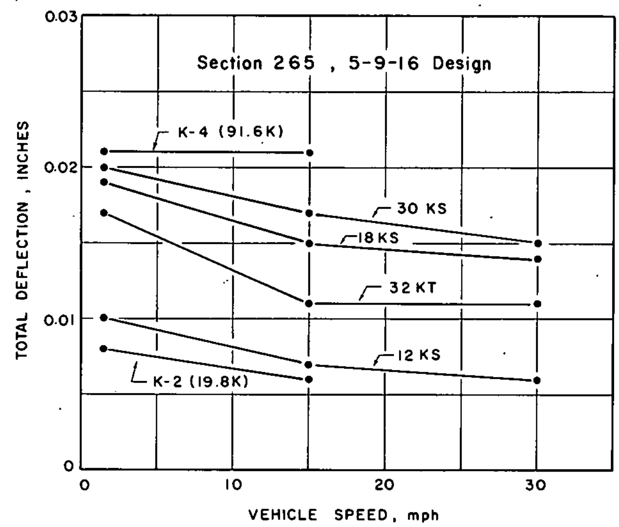


Figure 61. Relationship between vehicle speed and deflection.

TABLE 66

VEHICLE CHARACTERISTICS

Vehicle	Gross Vehicle Weight (kips)	Tread Contact (in.)			Unit Ground Pressure (psi)	Combat Equipped Weight (kips)	Over-All Vehicle Length (in.)
		Length	Width	Out-to-Out Width			
T–113	19.8	105	15	100	22.6	7.3	191.5
M–47	91.6	152.5	23	133	97.2	13.3	250.5

TABLE 67

INNER WHEELPATH DEFLECTIONS, FLEXIBLE SECTION 265 (5–9–16 DESIGN)
Deflection (10^{-3} in.)

Vehicle	Axle Load (kips)	Total			Embankment			Subbase		
		Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph	Creep	15 Mph	30 Mph
K–2[1]	19.8[2]	8	6	—	4	3	—	3	1	—
K–4[3]	91.6[2]	21	21	—	7	7	—	4	4	—
Conv.	12 S	10	7	6	7	5	3	4	2	1
Conv.	32 T	17	11	11	7	5	4	5	2	2
Conv.	18 S	19	15	14	9	5	5	5	3	3
Conv.	30 S	20	17	15	8	7	6	3	2	2

[1] T–113 personnel carrier.
[2] Gross vehicle load.
[3] M–47 tank.

TABLE 68

TRANSMITTED EMBANKMENT PRESSURE VALUES, TRANSVERSE DISTRIBUTION, SECTION 581 (5–6–12 DESIGN)

Vehicle and Load	Creep Speed		15 Mph	
	Distance from Gage (in.)	(psi)	Distance from Gage (in.)	(psi)
K–2 19.8[1]	15	1.40	16	1.20
	11	1.80	13	1.60
	7	2.10	6	2.20
	6	2.20	5	2.00
	2	2.30	—	2.00
	—	2.40	1	2.00
	4	1.90	4	1.80
	6	1.90	7	1.60
	9	1.60	10	1.30
	11	1.20	15	1.00
	17	0.70	17	1.00
	18	0.70	24	0.04
	24	0.30	26	0.03
	25	0.30		
K–4 91.6[1]	39	0.90	39	0.90
	34	1.70	34	1.40
	21	4.90	19	5.20
	20	5.50	18	5.20
	18	5.50	4	8.30
	3	9.40	1	9.30
	1	9.20	16	3.10
	4	8.60	21	1.70
	15	3.70	36	0.30
	22	2.00	39	0.20
	35	0.40		
	37	0.30		
	40	0.20		
Conv. 30 KS	28	2.90	25	3.20
	26	3.20	23	3.40
	17	6.20	15	7.10
	11	8.70	12	7.80
	1	10.10	7	8.40
	8	8.40		
	10	8.20		
	19	3.00		
	22	2.20		
	33	0.60		
Conv. 18 KS	22	3.20	11	3.70
	19	3.65	8	4.30
	17	4.00	7	3.40
	12	4.70	2	6.10
	12	4.60	—	6.30
	3	5.70	2	4.60
	2	5.60	4	4.10
	1	5.90	9	3.40
	—	7.12	16	2.00
	3	5.00	29	0.60
	5	6.80		
	12	3.90		
	13	3.60		
	14	3.30		
	25	1.10		
	26	1.10		
Conv. 12 KS	28	1.40	24	1.60
	27	1.60	22	1.60
	23	2.10	16	2.30
	16	2.90	11	2.80
	12	3.60	10	2.10
	9	3.60	1	3.20
	4	4.20	0	3.40
	1	4.10	8	2.30
	1	4.20	10	2.00
	10	2.70	13	1.50
	13	1.70	22	0.60
	14	1.60	23	0.60
	20	0.70	26	0.40
	22	0.60		
	25	0.50		

[1] Gross vehicle load.

TABLE 69

DYNAMIC STRAINS AND DEFLECTIONS, SECTION 389 (9.5–6 DESIGN)

Vehicle	Axle Load (kips)	Strain (10^{-6} in./in.)						Deflection (10^{-3} in.)		
		Compressive			Tensile					
		Creep Speed	15 Mph	30 Mph	Creep Speed	15 Mph	30 Mph	Creep Speed	15 Mph	30 Mph
K–2[1]	19.8[2]	9	8	—	—	—	—	19	16	—
K–4[3]	91.6[2]	22	21	—	—	—	—	62	59	—
Conv.	12 S	18	12	11	7	6	4	15	11	9
Conv.	32 T	17	18	17	12	12	12	20	19	19
Conv.	18 S	22	19	19	13	12	11	30	25	20
Conv.	30 S	32	31	25	16	15	13	45	38	32

[1] T–113 personnel carrier.
[2] Gross vehicle load.
[3] M–47 tank.

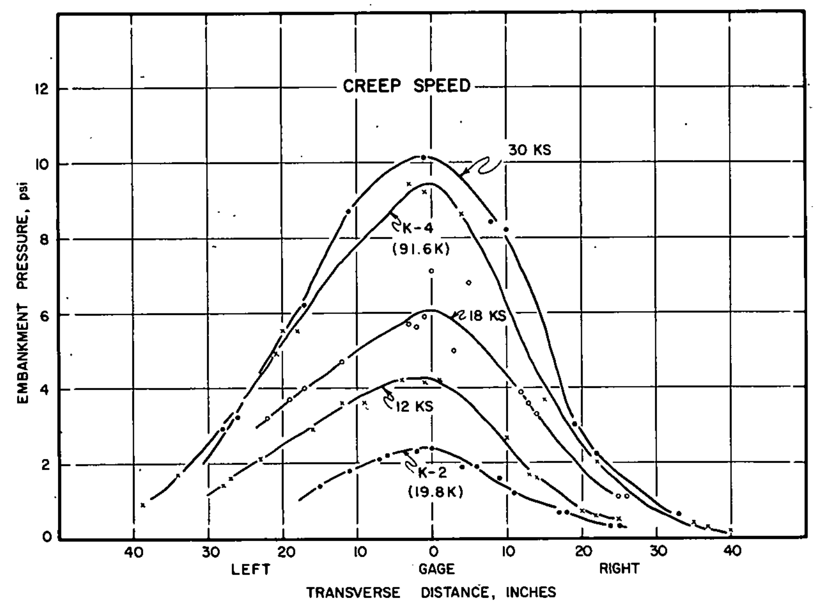


Figure 62. Transverse profile, embankment pressure, Section 581 (5-6-12 design).

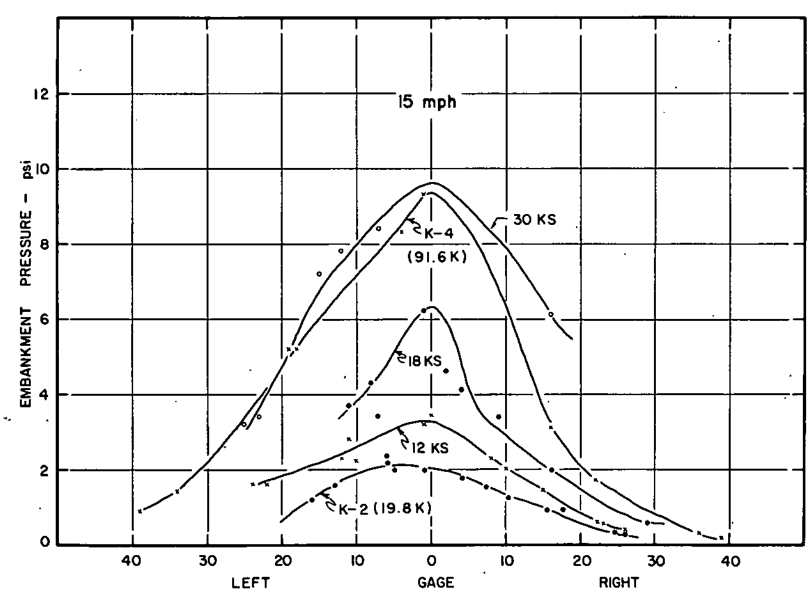


Figure 63. Transverse profile, embankment pressure, Section 581 (5-6-12 design).

developed in this section under the heavy tracked vehicle was nearly the same as that under the 30-kip single axle load.

7.3.2 Rigid Pavements

The same group of vehicles was operated over dynamic strain and deflection gages in the rigid section. Table 69 lists the strains and deflections recorded. No values of tensile strain were recorded for the track-laying equipment.

Figure 64 shows the relationships of vehicle speed and compressive strain and of vehicle speed and deflection, respectively. An increase in vehicle speed was associated with a decrease in deflection and strain, as was observed for the other studies reported.

7.4 NEEDED RESEARCH

The results of this study on the Road Test indicated that direct comparison between conventional tire units and track-laying equipment limited to dynamic edge strains, deflections or embankment pressures would not be of any great value. Therefore, further research is indicated in which studies should be made of the types and locations of dynamic measurement (and necessary instrumentation) that will prove effective for comparisons of track-laying and conventional equipment.

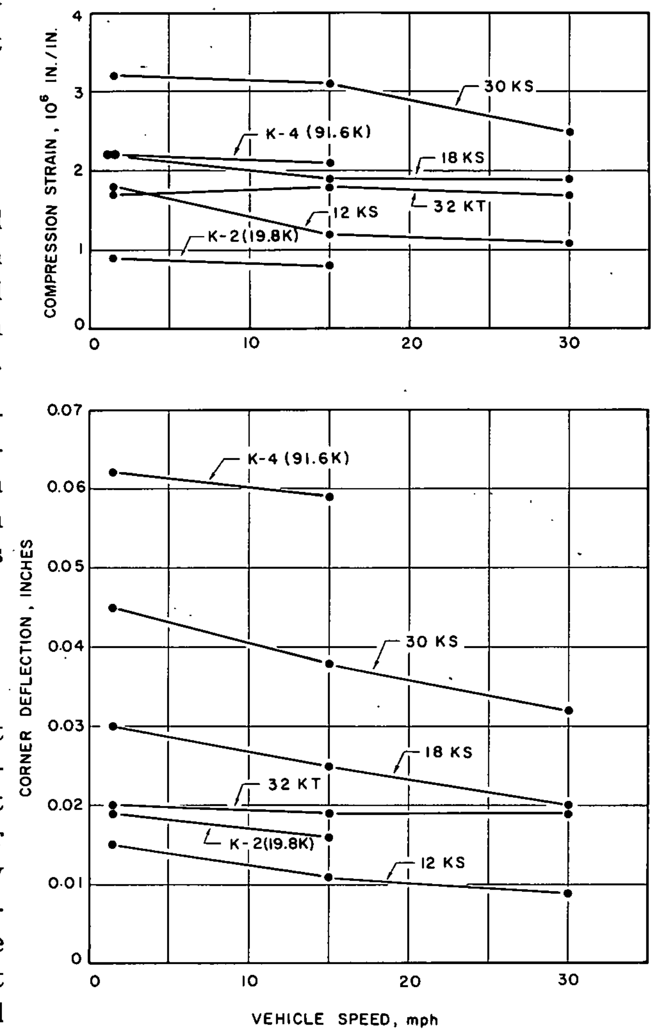


Figure 64. Relationship between vehicle speed and corner deflection and compressive strain, Section 389 (9.5-6 design).

Chapter 8

Braking, Impact, and Acceleration Study

8.1 SUMMARY

This study was designed to investigate means of determining the dynamic effects on pavements, bridges and vehicle cargoes of a selected group of vehicles when subjected to external accelerations.

A small pilot study to determine the effect of braking forces on the dynamic measurements of strain and deflection indicated that the location of the instrumentation and the design of the pavement sections available at the Road Test would not provide adequate information from which to draw conclusions. Consequently no further effort was made to determine the effect of braking forces.

Each of the several vehicles selected for this study was operated over ramps and dropped on the pavement. Simultaneously pavement strains, deflections and embankment pressures and vehicle and cargo accelerations were recorded.

Impact Study

In general, an increase in height of ramp caused an increase in deflection and embankment pressures at creep speeds. It did not, however, cause an appreciable increase in edge strains.

TABLE 70

BRAKING AND IMPACT STUDY VEHICLES

Vehicle Number	Axle Load (kips)	Tire Size (in.)	Tire Pressure (psi)
Conv.	18 S	10.00x20	80
Conv.	24 T	7.50x20	80
Conv.	32 T	9.00x20	80
Conv.	40 T	11.00x20	80
64	32 T	11.00x20	75
61	32 T	9.00x20	75
LPLS	32 T	46.00x24	35
GOER	13.4	29.5 x25	20
	23.4		25
	24.9		30
	35.1		35
HETAG	21.9	14.00x20	90
	21.9		
	16.8		
	17.0		
	16.5		

The rate of increase in either deflection or embankment pressure was not uniform for increase in height of ramp or vehicle speed.

Dynamic Load Study

The instrumentation for measuring dynamic load effect was not designed for the shock received in the drop tests; therefore, certain limitations were placed on the study. However, the dynamic load effect did increase appreciably with increase in ramp height.

Accelerations in the Vehicle

A complete analysis of the analog acceleration records was not included as a part of this study. A summary of the data shows that the magnitude of either the vehicle or cargo accelerations was such that the significance of the vehicle speed, tire pressure or tire design could not be determined.

Vehicle and cargo accelerations recorded during the impact study indicated a definite relationship between vehicle speed and height of ramp. An increase in speed and ramp height was associated with an increase in both cargo and vehicle accelerations.

8.2 SCOPE

Vehicles from the special suspension, commercial construction and military groups were subjected to a series of impact forces. Measurements of dynamic deflections, strains, transmitted embankment pressures, vehicle and cargo accelerations, and dynamic load were recorded. Table 70 lists the vehicles included in this study.

A small pilot study of the effect of maximum braking effort on bridges and rigid pavements was attempted. The units were operated at 20 mph past the gage points, and recordings of dynamic strains and deflections were made. In subsequent passes, the vehicles were subjected to "panic" stops with the trailer axles at various longitudinal positions with respect to the gage points.

Drop tests were conducted to measure the effect of vehicle impact. The units were operated at either 2 or 3 levels of speed (depending upon vehicle capabilities) over ramps of different heights varying in ¾-in. increments to a maximum of 3 in. One flexible section

(581, Loop 4) and one rigid section (389, Loop 6) were selected for the study.

Accelerometers were attached to the cargo directly over the rear axle and to the axle housing. During the drop tests recordings of acceleration were taken simultaneously with recordings of the change in tire pressure to determine the dynamic load effect.

8.3 BRAKING STUDY

The braking study was conducted on a very limited scale on one of the post-tensioned structures (Bridge 6B) in Loop 5 and on a 5-in. rigid pavement section with 9-in. subbase in Loop 3 (section 219). It proved to be difficult to predetermine the point at which the rear wheels would stop. Furthermore, the study was hampered by brake fatigue and by instrumentation failures.

The analog records of strain and deflection in the bridges failed to indicate any significant effect on the structures of the forces caused by the sudden braking of the vehicle. Similarly, the recordings for the strain and deflection of the rigid section failed to show any appreciable difference from those taken during a normal pass of the vehicle.

The failure of this study to show significant effects on these dynamic measurements can be partially attributed to a number of factors: lack of sensitivity in the instrumentation, the damping effect of the load transfer devices near the gage points and the damping effect of the bridge structure.

The measurements of changes in dynamic load effect and vehicle accelerations during the braking study were curtailed because early tests indicated the likelihood of damage to the recording equipment which was attached to the test vehicle.

8.4 FLEXIBLE PAVEMENT IMPACT STUDY

Table 71 gives the values of deflection from the drop tests for some of the special suspension and conventional vehicles. Difficulty was encountered in positioning the ramps with re-

TABLE 71

FLEXIBLE PAVEMENT TOTAL DEFLECTIONS[1] CREEP SPEED, SECTION 581 (5–6–12 DESIGN)

Vehicle Number	Axle Load (kips)	Deflection (10^{-3} in.)				
		0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp
61	32 T	24	31	30	35	35
64	32 T	26	21	22	21	16
LPLS	32 T	31	28	24	30	30
Conv.	32 T	24	25	30	28	27
Conv.	18 S	27	30	33	35	35
Conv.	24 T	24	24	20	18	19
Conv.	40 T	28	26	30	28	26

[1] Rear axle of tandems except vehicle No. 64.

TABLE 72

FLEXIBLE PAVEMENT TOTAL DEFLECTIONS, GOER AND HETAG, SECTION 581 (5–6–12 DESIGN)

Vehicle Number	Axle Load (kips)	Speed (mph)	Deflection (10^{-3} in.)				
			0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp
GOER	23.4 S	Creep	35	44	44	50	52
	13.4 S		23	31	31	37	35
	30.2 S		50	47	52	57	60
	25.8 S		41	38	40	50	52
HETAG	21.9 S		35	34	28	27	26
	21.9 S		38	36	32	30	31
	16.8 S		27	28	23	24	23
	17.0 S		31	31	25	27	26
	16.5 S		29	28	23	25	27
GOER	23.4 S	15	29	17	23	54	57
	13.4 S		16	8	14	30	28
	30.2 S		38	38	36	52	48
	25.8 S		37	26	24	40	44
HETAG	21.9 S		27	21	21	20	19
	21.9 S		27	20	18	21	23
	16.8 S		19	19	20	21	20
	17.0 S		23	19	13	18	16
	16.5 S		23	18	19	18	21

spect to the gage point to obtain maximum effect. Thus, the values for speeds greater than creep were not usable and are not reported. The values are shown for the rear trailer axle of each vehicle, except for vehicle 64. The values for this unit are not directly comparable to those for other units with the same axle load because of the unique axle-wheel arrangement.

A study of the data failed to indicate any definite trend of deflection with height of ramp or any meaningful variation among vehicles. In general a moderate increase was noted as ramp height increased, but the variability of the data did not permit any statement of significant effect.

Table 72 gives the values of deflection for the military units subjected to the drop tests. The GOER was operated at two different loads and two speeds, while the HETAG was operated at one load and two speeds. The data for the GOER at creep speed appear rational and show a highly significant increase in deflection with an increase in height of ramp for both load conditions. However, the values of deflection for five axles of the HETAG are highly variable and show no trend with change in ramp height or vehicle speed.

TABLE 73

EMBANKMENT PRESSURES UNDER DROP TESTS, CREEP SPEED, SECTION (5–6–12 DESIGN)

Vehicle Number	Axle Load (kips)	Pressure (psi)				
		0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp
61	32 T	5.55	5.25	6.55	6.85	7.4
64 [1]	32 T	4.35	4.58	4.40	4.63	4.63
LPLS	32 T	6.30	6.55	6.45	7.10	7.35
Conv.	32 T	5.60	5.70	5.90	6.00	6 30
Conv.	18 S	4.90	5.60	5.75	6.50	6.95
Conv.	24 T	3.90	3.80	3.80	4.10	4.50
Conv.	40 T	5.30	5.00	5.25	5.50	5.75

[1] For single wheel load of 4 kips.

TABLE 74

EMBANKMENT PRESSURE RATIO, RAMP/NO RAMP, CREEP SPEED, SECTION 581 (5–6–12 DESIGN)

Vehicle Number	Axle Load (kips)	Ratio				
		0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp
61	32 T	1.00	0.95	1.18	1.23	1.33
64	32 T	1.00	1.06	1.01	1.07	1.07
LPLS	32 T	1.00	1.04	1.02	1.13	1.17
Conv.	32 T	1.00	1.02	1.05	1.07	1.13
Conv.	18 S	1.00	1.14	1.18	1.33	1.42
Conv.	24 T	1.00	0.97	0.97	1.05	1.15
Conv.	40 T	1.00	0.94	0.99	1.04	1.08

TABLE 75

EMBANKMENT PRESSURES, DROP TEST, HETAG AND GOER, SECTION 581 (5–6–12 DESIGN)

Vehicle Number	Axle Load (kips)	Vehicle Speed (mph)	Pressure (psi)				
			0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp
GOER	23.4 S	Creep	7.30	8.40	8.80	9.10	9.40
	13.4 S		5.45	6.00	6.25	5.95	6.00
	30.2 S		10.70	9.60	10.35	11.05	12.05
	25.8 S		8.70	8.30	9.10	10.50	10.35
HETAG	21.9 S		5.40	4.80	4.65	4.30	4.65
	21.9 S		5.80	5.70	5.10	5.10	5.25
	16.8 S		4.50	4.35	3.90	3.65	4.00
	17.0 S		4.55	4.55	4.10	4.00	4.30
	16.5 S		4.65	4.30	4.10	4.05	4.60
GOER	23.4 S	15	7.50	7.15	7.30	9.70	10.95
	13.4 S		4.70	5.45	6.05	6.25	7.20
	30.2 S		8.95	7.90	5.70	12.30	14.80
	25.8 S		8.00	6.05	6.25	11.30	12.05
HETAG	21.9		4.65	4.65	3.90	3.10	2.30
	21.9		5.25	5.00	6.15	8.15	8.85
	16.8		4.30	4.90	4.25	3.10	2.15
	17.0		4.75	5.10	5.80	6.90	6.25
	16.5		4.15	4.20	4.75	7.65	5.50

Figure 65. GOER operated over ramps in drop tests at rigid pavement instrumentation van.

Measurements of the pressures transmitted to the embankment were taken on section 581, Loop 4 during the impact tests under the same group of vehicles. Table 73 gives data from these tests for the special suspension vehicles. As before, only the data taken at creep speed are shown. Data from tests at higher speeds were highly scattered. The date appear to be rational, and a definite increase was noted in embankment pressure with height of ramp. Again, vehicle 64 cannot be compared directly to the other vehicles of equal load.

The ratios of embankment pressure at different drop heights to the embankment pressure at no drop are given in Table 74.

The embankment pressures noted under the military vehicles are given in Table 75. The positioning of the ramp with respect to the gage point did not appear to be critical for the GOER, and the higher speed data for this unit are shown. The data obtained for the GOER show a clear effect of height of ramp on the measured embankment pressures at both levels of speed; however, the data for the HETAG

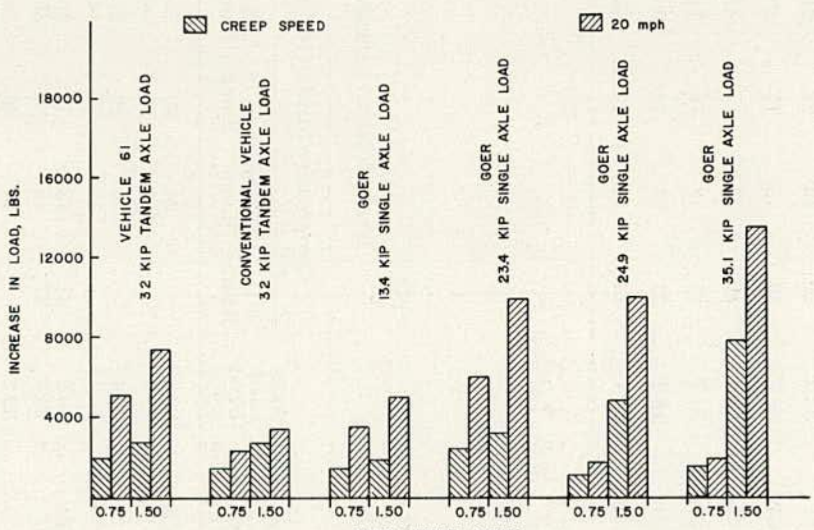


Figure 66. Increase in effective axle load with increase in height of ramp and vehicle speed.

TABLE 76

RIGID PAVEMENT OBSERVATIONS, DROP TEST, SECTION 389 (9.5–6 DESIGN)

Vehicle Number	Axle Load (kips)	Compressive Strain (10^{-6} in./in.)					Tensile Strain (10^{-6} in./in.)					Deflection (10^{-3} in.)				
		0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp	0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp	0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp
61	32 T	20	19	21	21	22	13	12	13	14	13	31	30	33	76	40
64	32 T	14	16	17	20	16	11	11	11	12	14	22	26	29	73	36
LPLS	32 T	20	18	17	17	16	11	11	12	13	14	28	26	28	92	36
Conv.	32 T	21	18	18	21	21	8	8	8	8	10	26	26	27	28	40
Conv.	18 T	21	20	20	23	21	9	9	10	10	13	19	20	22	67	34
Conv.	24 T	14	14	14	15	17	8	8	8	9	9	20	21	24	28	34

TABLE 77

RIGID PAVEMENT OBSERVATIONS, DROP TEST, HETAG AND GOER, SECTION 389 (9.5-6 DESIGN)

Vehicle Number	Axle Load (kips)	Compressive Strain (10^{-6} in./in.)					Tensile Strain (10^{-6} in./in.)					Deflection (10^{-3} in./in.)				
		0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp	0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp	0-In. Ramp	0.75-In. Ramp	1.50-In. Ramp	2.25-In. Ramp	3.00-In. Ramp
GOER	23.4		24	23	24	23		7	7	7	7		34	31	28	27
	13.4		14	14	14	14		7	7	7	7		22	20	19	19
	23.4		24	20	23	19		6	8	7	7		35	36	38	38
	13.4		15	11	14	15		6	8	7	7		22	22	22	19
GOER	30.2	31	32	32	30	29	11	10	9	11	10	32	33	33	36	27
	25.8	27	25	26	25	27	11	10	9	11	10	28	27	28	31	33
	30.2		33	29	30	33		10	9	11	12		34	32	44	52
	25.8		26	25	23	23		10	9	11	12		29	31	33	39
HETAG	17.0	22	18	20	17	19	9	10	12	10	9	39	39	49	43	36
	16.5	12	11	13	13	13	9	10	12	10	9	38	36	48	44	38
	17.0	20	25	30	23	33	9	11	15	18	15	36	42	58	59	51
	16.5	12	11	13	13	13	9	11	15	18	15	34	42	54	58	50

appear to be highly variable and irrational at either level of speed.

8.5 RIGID PAVEMENT IMPACT STUDY

Table 76 lists the values for strain (compressive and tensile) and slab deflection for the special suspension and for the conventional vehicles. These data for strains failed to show any trend with respect to the height of ramp. With few exceptions, the values for slab corner deflection increased noticeably with increase in ramp height. The values for the deflections at a ramp height of 2.25 in. are unexplainably large for some of the vehicles.

Figure 65 is a view of the drop tests.

The values of strain and deflection for the GOER and the HETAG are given in Table 77 for two levels of speed. Very little change in strain can be associated with change in height of ramp; however, the values for deflection show a slight increase with an increase in ramp height.

The lack of significant trends in these data for the rigid section can be explained partially by the fact that the other axles of the vehicles were either on the slab or ramp at the time the effect of an impact was being measured.

8.6 DYNAMIC LOAD STUDY

Changes in dynamic load effect were recorded in a limited number of vehicles during the drop tests. The studies could not be continued because of instrument failures. Since the instruments were mounted in a trailer towed by the vehicle under test, they were run over the ramps along with the test vehicle. The shock was greater than that for which they had been designed.

Table 78 gives the mean increase in effective axle load for the different ramp heights for the vehicles included in the study. Each value is the mean of three field recordings at speeds of creep and 15 mph. The total effective load increase for a tandem axle would be twice the value shown.

Figure 66 shows the relationship of vehicle speed and ramp height on the effective axle load. The increase in effective axle load is appreciable for changes in ramp height and/or vehicle speed, but a more detailed investigation appears necessary before definite relationships can be shown.

8.7 ACCELERATIONS IN THE VEHICLE

A series of acceleration studies was conducted on the special sections in Loop 6 (described in Chapter 3 Section 3.6). All of the tire pressure-tire design and special suspension vehicles, in addition to some of the commercial construction and military equipment, were operated over the sections at speeds

TABLE 78

INCREASE IN EFFECTIVE AXLE LOAD[1]

Vehicle Number	Axle Load (kips)	Ramp Height (in.)	Load Increase (lb)	
			Creep	20 Mph
61	32 T	0.75	2,070	5,230
		1.50	2,830	7,510
		2.25	3,220	10,150
		3.00	3,400	11,330
Conv.	32 T	0.75	1,580	2,370
		1.50	2,810	3,490
GOER	13.4 S	0.75	1,510	3,640
		1.50	1,920	5,140
	23.4 S	0.75	2,510	6.160
		1.50	3,250	9,930
	24.9 S	0.75	1,170	4,880
		1.50	1,780	10,080
	35.1 S	0.75	1,580	7,880
		1.50	1,970	13,630

[1] Total increase in load over static load for a tandem axle vehicle would be twice the values listed. Each value is the mean of three field recordings.

TABLE 79

CARGO AND AXLE ACCELERATIONS DROP TEST

Vehicle Number	Axle Load (kips)	Vehicle Speed (mph)	Ramp Height (in.)	Acceleration (g's)	
				Axle	Cargo
61	32 T	Creep	0	0.10	0
			0.75	0.20	0.10
			1.50	0.22	0.20
		15	0	0.20	0.10
			0.75	1.40	0.40
			1.50	1.80	0.55
		20	0	0.27	0.15
			0.75	1.55	0.38
			1.50	2.40	0.75
Conv.	32 T	Creep	0	0	0
			0.75	0	0.10
			1.50	0.10	0.10
		15	0	0	0
			0.75	1.15	0.45
			1.50	1.75	0.45
		20	0	0.20	0.20
			0.75	1.15	0.50
			1.50	1.90	0.70
GOER	23.4 S 13.4 S[1]	Creep	0	—	0
			0.75	—	0
			1.50	—	0.10
		15	0	—	0
			0.75	—	0.20
			1.50	—	0.30
		20	0	—	0
			0.75	—	0.30
			1.50	—	0.60
GOER	35.1 S 24.9 S[1]	Creep	0	—	0
			0.75	—	0
			1.50	—	0
		15	0	—	0
			0.75	—	0.10
			1.50	—	0.20
		20	0	—	0
			0.75	—	0.30
			1.50	—	0.50

[1] Rear axle accelerometer mounted on cargo directly above this axle.

of 10 and 30 mph. A 10.0-g accelerometer was rigidly attached to the vehicle axle, and a 5.0-g accelerometer was attached to the vehicle cargo directly above the trailer axle.

For all of the tire pressure-tire design and special suspension vehicles, the range of accelerations noted on either the axle or cargo was from 0.3 to 0.5 g, with a mean value slightly greater than 0.3 g. In only two instances were values lower than 0.3 g recorded. The LPLS unit and vehicle 61 indicated cargo accelerations lower than 0.3 g at 10 mph on the smooth test section.

The axle arrangement for the GOER was such that only the 5-g accelerometer on the cargo could be installed. Cargo accelerations for the GOER were lower than 0.3 g at the heavier loads (35.1 and 24.9 kips) and higher inflation pressures (35 and 30 psi). Accelerations of greater than 0.3 g were recorded at the lighter loads (23.4 and 13.4 kips) and lower inflation pressures (25 and 20 psi) on both test sections at speeds of 10 and 20 mph.

The accelerations recorded for the medium scraper unit with axle loads of 52.5 and 35.0 kips and tire inflation pressure of 45 psi (normal) were equal to or less than 0.3 g for the cargo and about 0.5 g for the axle at vehicle speeds of 10 and 20 mph on both test sections.

The accelerations reported for all units in this portion of the study are low, possibly because of the high serviceability of the sections selected.

Before the failure of the recording equipment a small number of accelerations of cargo and axle were recorded on vehicles subjected to the drop tests on Loops 4 and 6. Table 79 gives these data for three vehicles for various heights of ramp and for several vehicle speeds. Each of the values is the mean of at least three tests, and represents the maximum values of acceleration recorded in each test.

Figure 67 shows the relationship of cargo and axle accelerations to vehicle speed and height of ramp. Ramp heights greater than 1.5 in. were not used for these vehicles (except vehicle 61). Both speed and height of ramp show appreciable effects on the acceleration of the vehicle and of the cargo.

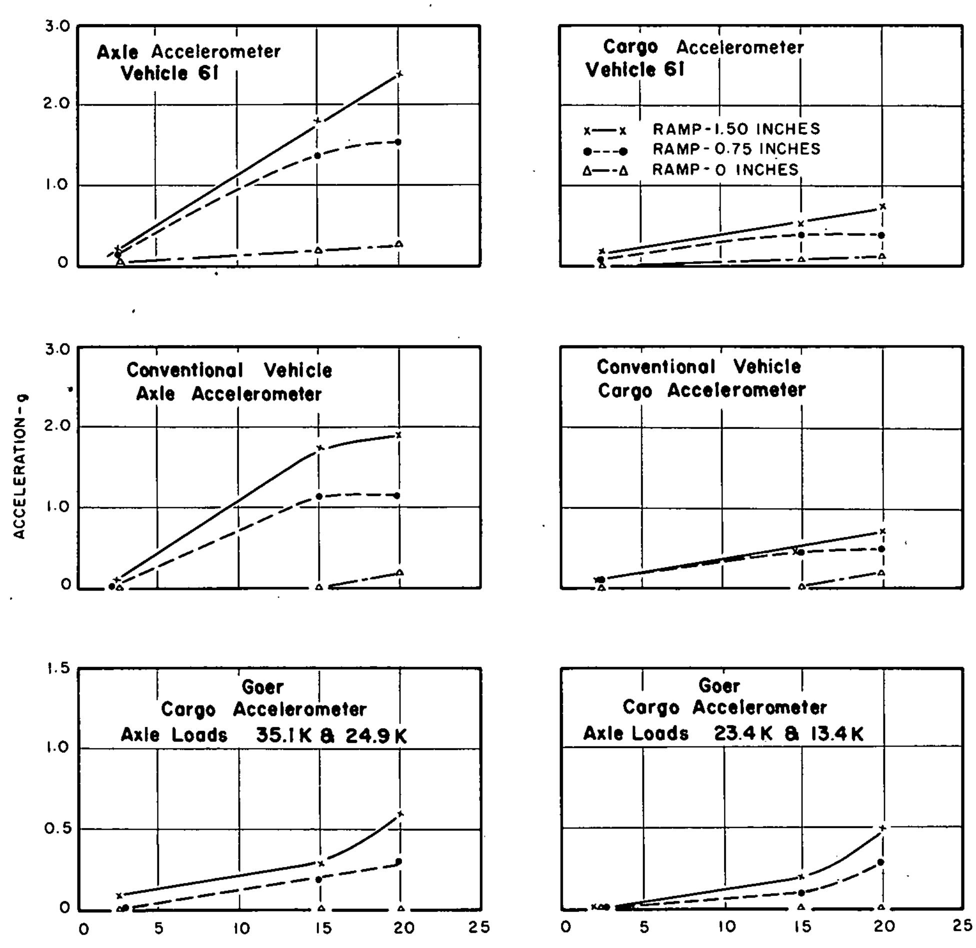


Figure 67. Relationship between cargo accelerations and vehicle speed, drop test.

8.8 NEEDED RESEARCH

The general trends reported for this study indicate the need for further investigation of the dynamic effect of vehicles on the pavement and bridges when subjected to external accelerations. These investigations should include a greater range of pavement serviceabilities, vehicle speeds and instrumented pavements than were available at the Road Test at the time of this study.

An investigation of the vehicles and their characteristics under various load conditions and tire pressures would make possible a more detailed analysis of the vehicle and cargo accelerations and dynamic load effect data.

As noted previously, a performance study of pavement sections under the various vehicle configurations being investigated by the Department of Defense with dynamic measurements is needed before unqualified statements can be made concerning effects of vehicle design on pavement and cargo.

Chapter 9

Bridge Tests with Increasing Loads

9.1 OBJECTIVE AND SCOPE

Test bridges which survived tests with repeated high overstress were utilized in a study of flexural bridge capacity. This study is described in detail in Section 3.6 of AASHO Road Test Report 4, Bridge Research, and is reported here only in summary form.

Ten bridges were included in the study. Table 80 shows the bridge types and the governing design stresses. Four bridges had steel beams (1A, 3B, 9A and 9B), four had prestressed concrete beams (5A, 5B, 6A and 6B), and two had reinforced concrete beams (8A and 8B).

All bridges were simple span structures consisting of three beams and a reinforced concrete slab. The beams were supported on 50-ft spans. The slabs were 15 ft wide.

Because of differences in design criteria, direct comparisons cannot be made between the steel, prestressed concrete and reinforced concrete structures.

The ten bridges were subjected to passages of vehicles having successively heavier loads. Each load was applied 30 times, and testing was discontinued when the slab was crushed, when the tension steel was fractured, or when an already extreme permanent set continued to increase at an increasing rate with each passage of the test vehicle.

It should be noted that all ten bridges had previously been subjected to approximately 557,000 stress repetitions caused by the regular test traffic. In addition, Bridges 5B, 6A, 6B and 8A had been subjected to accelerated fatigue tests which increased the number of stress cycles to approximately 1,500,000 as described in Report 4.

The tests with increasing loads were designed to study the response of the bridges to loads approaching their ultimate capacity, to determine the manner of failure under moving loads, and to provide data for checking ultimate strength theories.

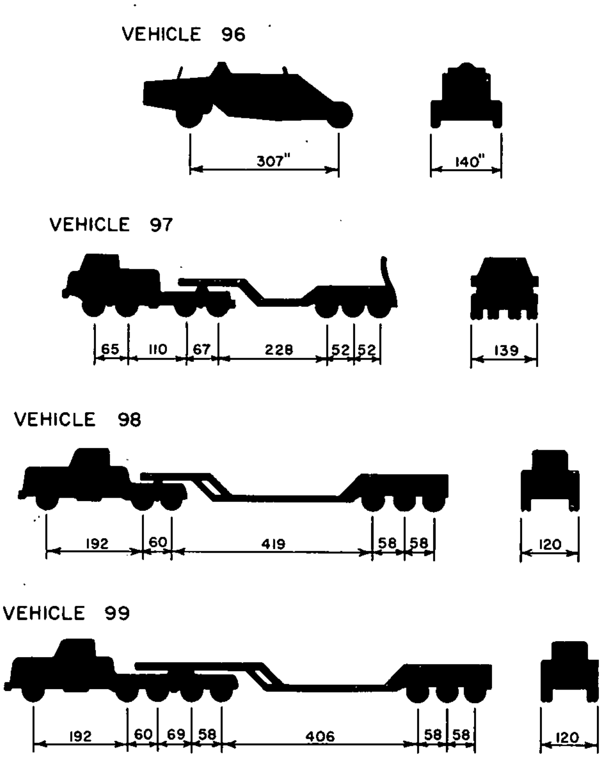


Figure 68. Special test vehicles.

TABLE 80

DESIGN OF BEAMS

Bridge Designation	Bridge Type	Governing Design Stress, Max. Tension (psi)
	(*a*) STEEL BRIDGES	
1A	Noncomposite with cover plate	27,025
3B	Composite with cover plate	26,940
9A, 9B	Noncomposite with cover plates	26,970
	(*b*) PRESTRESSED CONCRETE BRIDGES	
5A	Post-tensioned	820
5B	Post-tensioned	346
6A	Pretensioned	828
6B	Pretensioned	310
	(*c*) REINFORCED CONCRETE BRIDGES	
8A, 8B	Cast-in-place	30,900

TABLE 81

VEHICLE LOADS CAUSING FIRST SET OR DECREASE OF STIFFNESS

Bridge	Vehicle [1] Number	Axle Weight (kips)							
		1 [2]	2	3	4	5	6	7	8
				(a) STEEL BRIDGES					
1A	62 [3]	11.4	23.8	23.3	23.6	24.2	—	—	—
3B	99	17.0	21.9	21.9	27.5	27.9	32.9	32.3	32.3
9A	97	14.1	13.8	19.6	20.7	35.8	36.6	38.9	—
9B	97	14.1	14.2	16.9	17.0	21.8	23.2	23.0	—
				(b) PRESTRESSED CONCRETE BRIDGES					
5B	97	13.5	14.0	27.6	26.0	37.1	38.6	41.6	—
6A	97	14.5	13.4	18.4	19.7	28.3	30.0	32.5	—
6B	97	13.5	14.0	27.6	26.0	37.1	38.9	41.6	—
				(c) REINFORCED CONCRETE BRIDGES					
8A	97	13.6	13.7	24.2	24.7	37.5	41.2	47.8	—
8B	97	13.6	13.7	24.2	24.7	37.5	41.2	47.8	—

[1] Vehicles described in Figure 68.
[2] Front axle; remaining axles numbered consecutively from front to back.
[3] Regular test vehicle from lane 2, Loop 6.

Figure 68 shows the configuration and dimensions of the test vehicles. Table 81 gives the vehicle loads which caused the first substantial permanent set or decrease of stiffness in each bridge, and Table 82 is a summary of test results at that stage of the testing. Table 83 gives the maximum vehicle loads applied to each bridge, and Table 84 summarizes the results at that stage of the testing.

9.2 SUMMARY

The following is a summary of the tests on each type of bridge. Section 3.6 of Report 4 contains detailed descriptions of the behavior of the bridges plus moment-deflection and permanent deformation diagrams which provide a complete history of each test.

Steel Bridges

All steel beam bridges included in this study failed by yielding of the beams and accumulation of a large permanent set (more than 12 in.). Noncomposite steel bridges were tested until the permanent deformations increased at an increasing rate with each successive pass of the same load. On the one composite bridge, limitations on test vehicle capacity forced discontinuance of testing before the slab was crushed. However, at the conclusions of the test, the total permanent set at midspan exceeded 13 in. Figure 69 shows Bridge 1A at the end of the test.

TABLE 83

MAXIMUM VEHICLE LOADS

Bridge	Axle Weight [1] (kips)						
	1 [2]	2	3	4	5	6	7
			(a) STEEL BRIDGES				
1A	14.1	14.2	16.7	18.3	28.4	29.8	31.8
3B	13.8	13.9	31.5	30.6	72.3	73.5	82.3
9A	13.8	13.8	28.8	29.0	42.8	45.4	50.2
9B	14.2	14.4	29.3	30.7	44.8	46.5	51.2
			(b) PRESTRESSED CONCRETE BRIDGES				
5A	13.5	14.0	27.6	26.0	37.1	38.9	41.6
5B	13.8	13.9	31.5	30.6	72.3	73.5	82.3
6A	14.2	14.0	24.4	23.5	41.7	45.0	49.1
6B	13.8	13.9	31.5	30.6	72.3	73.5	82.3
			(c) REINFORCED CONCRETE BRIDGES				
8A	14.5	14.0	30.9	30.3	43.0	44.9	52.8
8B	14.5	14.0	30.9	30.3	43.0	44.9	52.8

[1] Vehicle 97 used in all tests.
[2] Front axle; remaining axles numbered consecutively from front to back.

TABLE 82

SUMMARY OF DATA AT FIRST SET OR DECREASE OF STIFFNESS

Bridge	Load Causing First Set or Decrease of Stiffness: Load No.	No. of Trips	Speed (mph)	Max. Static Moment (ft-kips): Midspan	Max. Static Moment (ft-kips): End Plate	Est. Impact[1] (%)	Live Load Defl. (in.)	Perm. Set (in.)	Total No. of Trips[2]	Total Perm. Set[2] (in.)	Mode of Behavior
(a) Steel Bridges											
1A	6	30	25	635	633	21	3.0	0.3	180	0.4	Yielding of steel
3B	3	30	30	1,060	920	13	1.3	0.3	69	0.4	Yielding of steel
9A	3	30	30	1,230	1,160	13	3.3	0.7	90	0.8	Yielding of steel
9B	3	30	30	1,230	1,160	9	2.9	0.4	90	0.5	Yielding of steel
(b) Prestressed Concrete Bridges											
5B	3	35	30	1,315	—	12	0.7	0	95	0	Cracking of conc.
6A	3	30	30	1,000	—	20	0.7	0	90	0	Cracking of conc.
6B	5	30	30	1,300	—	10	0.3	0	150	0	Cracking of conc.
(c) Reinforced Concrete Bridges											
8A	4	30	30	1,390	—	11	1.7	10.7	120	10.9	Yielding of steel
8B	4	30	30	1,390	—	13	1.7	5.9	120	6.1	Yielding of steel

[1] Based on strains.
[2] From beginning of tests with increasing loads.

TABLE 84

SUMMARY OF DATA AT MAXIMUM TEST LOAD

Bridge	Maximum Load							Total No. of Trips[1]	Total Perm. Set[1] (in.)	Mode of Failure
	No. of Trips	Nominal Speed (mph)	Max. Static Moment (ft-kips)		Estimated Impact (%)	Live Load Deflection (in.)	Perm. Set (in.)			
			Midspan	End Plate						
(a) Steel Bridges										
1A	13	20	1,000	900	15[2]	3.9	10.4	403	15.4	Permanent set
3B	14	Creep	2,520	2,330	0	3.2	5.5	214	14.1	Permanent set
9A	20	20	1,535	1,490	28[2]	4.4	10.6	140	13.2	Permanent set
9B	30	20	1,580	1,520	14[3]	4.0	5.9	190	12.3	Permanent set
(b) Prestressed Concrete Bridges										
5A	35	30	1,315	—	—	7.8	10.8	95	10.9	Concrete crushing
5B	70	Creep	2,520	—	0	8.5	7.3	285	8.1	Steel fracture
6A	18	20	1,500	—	20[2]	7.9	1.5	198	7.6	Steel fracture
6B	45	Creep	2,520	—	0	8.0	4.8	365	6.4	Steel fracture
(c) Reinforced Concrete Bridges										
8A	2	30	1,550	—	18[2]	3.2	4.3	122	15.2	Concrete crushing
8B	7	30	1,550	—	16[2]	3.0	8.4	127	14.5	Concrete crushing

[1] During tests with increasing loads.
[2] Based on deflections.
[3] Based on strains.

The tests on steel beam bridges helped to demonstrate that the composite bridge was clearly superior to the noncomposite bridges. The composite bridge carried a 67 percent higher moment at first set and 150 percent higher at ultimate load than did a noncomposite bridge with steel sections only about 10 percent weaker.

The ultimate strength of all four steel bridges, computed on the basis of the fully plastic stress distribution, was always less than the observed external moment to which the bridge was subjected at failure. The external moment exceeded the computed capacity by 2 to 27 percent at midspan and by 23 to 32 percent at the ends of cover plates.

Prestressed Concrete Bridges

Of the four prestressed concrete bridges in this study, three failed by fracture of the prestressing steel and one by crushing of the concrete slab following an apparent bond failure between the wires and the grout. The beams of the latter bridge had been thoroughly cracked by the regular test traffic.

The ultimate strength of all four prestressed bridges was estimated from accepted formulas developed from laboratory tests, assuming a fully bonded condition for three bridges and a fully unbonded condition for the bridge with cracked beams. The computed capacity was compared with the maximum static moment caused by the heaviest test vehicle. The ratio

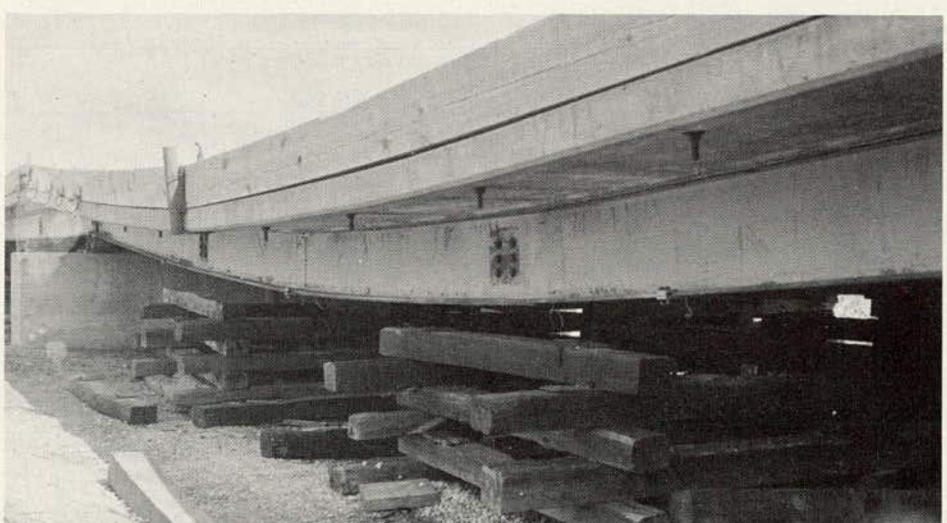


Figure 69. Bridge 1A at end of test.

of this moment to the computed capacity was 1.02 to 1.16 for the bonded case and 0.96 for the unbonded case.

Two of the prestressed concrete bridges had post-tensioned beams stressed with smooth wire and grouted, while two had pretensioned beams stressed with 7-wire strand. The tests with increasing loads helped to indicate that the bond provided by the strand was superior to that provided by the smooth wires embedded in grout. Figure 70 shows beams of Bridges 5B and 6B after failure.

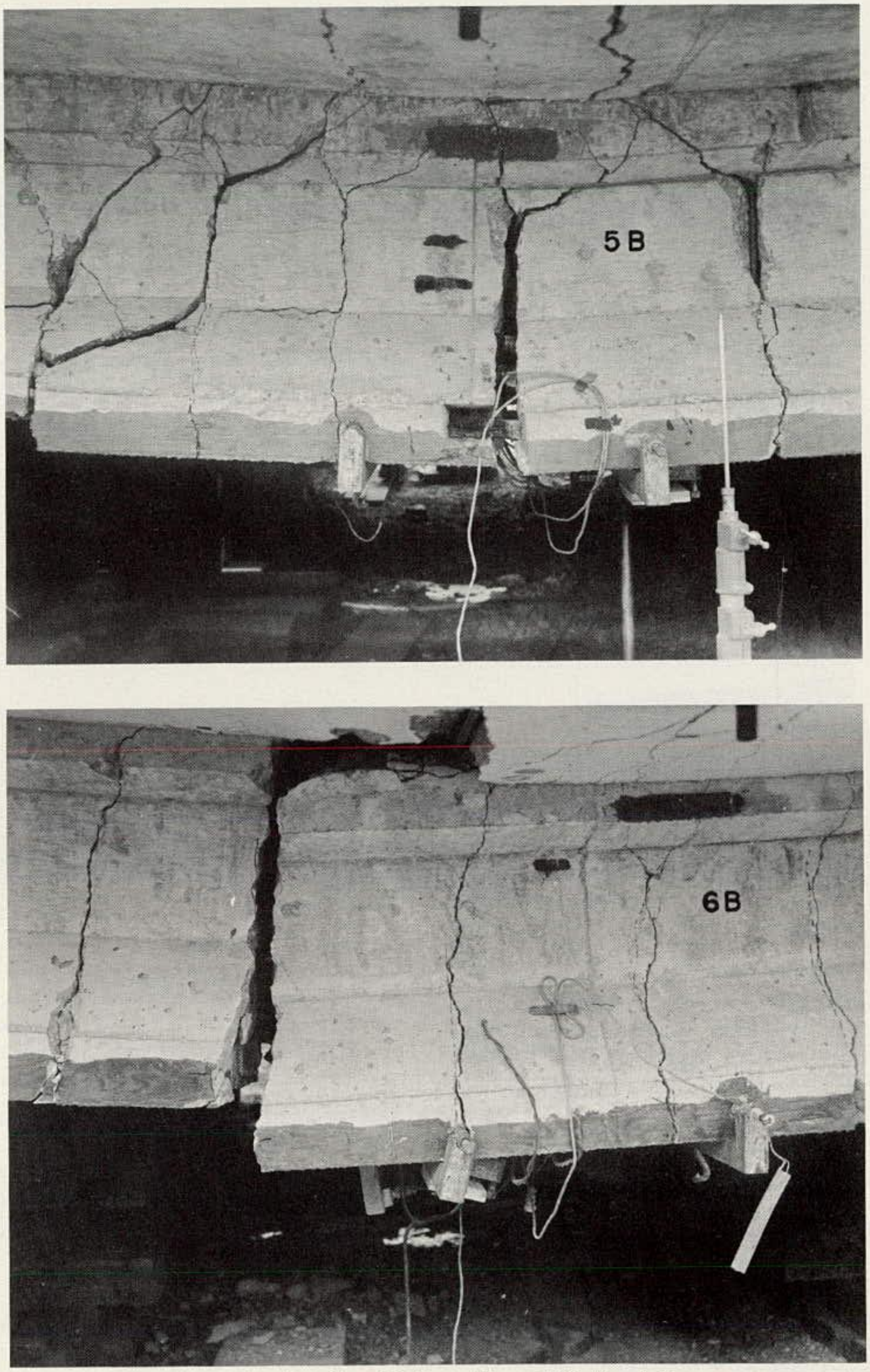


Figure 70. Details of Bridges 5B and 6B after failure.

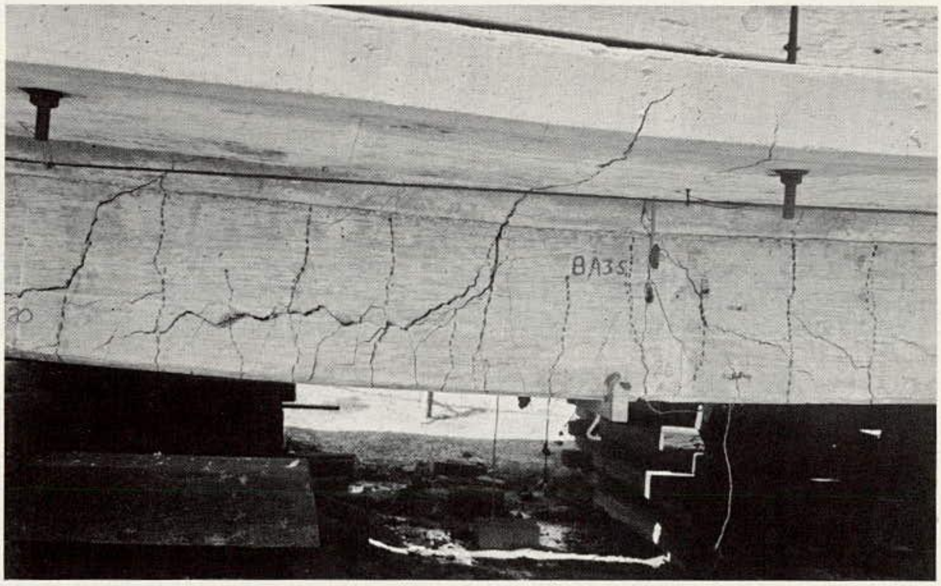


Figure 71. Bridges 8A and 8B after failure.

Reinforced Concrete Bridges

Both reinforced concrete bridges in this study failed by yielding of the tension reinforcement followed by crushing of the slab. The permanent set at failure exceeded 14 in. in both structures.

The ultimate strength of the bridges was computed from accepted formulas developed from laboratory tests. The maximum static moment caused by the heaviest vehicle exceeded the computed strength by 4 percent. Figure 71 shows Bridges 8A and 8B after failure.

Chapter 10

Special Studies During Research Phase

This chapter has been included to call attention to several experiments conducted at the AASHO Road Test but not related directly to the project's major objectives. Some of these experiments were conducted by Road Test personnel, others by outside agencies at the request of and with the cooperation of Road Test personnel, and still others by outside agencies primarily for their own benefit. The existence of the Road Test physical plant, constructed under highly controlled conditions and subjected to exactly known traffic loadings, made it an ideal testing ground for these side studies. Nearly all of these side studies were large enough to warrant independent publication. Where this was the case references to the source of publication are given.

10.1 DEVELOPMENT OF NUCLEAR TESTING EQUIPMENT

The project staff undertook an extensive instrumentation development program for the measurement of density-in-place of the various layers of the pavement structure at the Road Test. The work of previous investigators was extended to produce field equipment and the techniques for using it. Since this work was completed, commercial equipment has been placed on the market which, in some respects, is superior to that developed at the project.

Two papers were given by staff men. The first, which appeared in HRB Special Report 38, described the development of the Road Test equipment and some of the problems associated with the nuclear system of density measurement. The second, given at the American Society for Testing Materials, Symposium of Nuclear Surface Density and Moisture Tests, June 1960, described techniques for evaluation of nuclear equipment. In the second paper comparisons are made between Road Test equipment and commercial equipment.

In a broad sense, these studies indicated that the nuclear equipment has a great potential in the control of construction moisture and density. Properly calibrated, the nuclear systems are probably superior to conventional techniques for measuring in-place density. They are certainly much easier to use than conventional equipment. The Road Test investigators found, however, that the calibration relationships for the equipment depended upon the particular materials to be tested. No way was found to cut short a rather tedious calibration procedure.

10.2 VOLUMETRIC DETERMINATION OF WESTERGAARD FOUNDATION MODULUS

The U. S. Army Corps of Engineers, Ohio River Division Laboratories, has developed a unique technique for the determination of k to be used in the Westergaard equations for stress. Measurements are made from the top of the rigid pavement slab. The value of k recorded is said to be that of the foundation beneath the slab.

Engineers from the Ohio River Division Laboratories conducted several of these tests on the pavements of the AASHO Road Test. Their preliminary analysis of the data indicated that the k-values obtained by the volumetric technique are very similar to those obtained from plate tests performed by the method normally used by the Corps of Engineers. Final analyses of the data had not been completed. Presumably when completed, they will be made available upon request to the Director of the Laboratories at Mariemont, Cincinnati 27, Ohio. A description of the test procedure, furnished by the Laboratories, along with some information bearing on the development of the test follows. The data collected on the Road Test are available in Data System No. 4151.

10.2.1 Description of Test Procedure

General.—A known load is applied to the surface of the pavement and the total volumetric vertical displacement of the pavement is measured. The volumetric k-value is the load divided by the volume displaced by that load.

Load.—Static load is applied to the pavement through an 18-in. diameter plate. The plate must be level and firmly seated on a minimum thickness of sand to assure uniform bearing. The load should be applied in three or more increments with the maximum load producing a vertical movement of the plate of not less than 0.040 in. and not more than 0.060 in.

Vertical Displacements.—The vertical displacement of the concrete is measured along two lines at right angles to each other. These measurements are made with 0.001-in. dial extensometers. The first dial is located 1 ft from the center of the plate with additional dials spaced at 2-ft intervals for a distance of at least 11 ft from the center of the loaded area. The beam holding the dials may be cantilevered and supported on the pavement as long as the supports are at least 4 l (Westergaard's l) distance from the center of the applied load.

Note of Caution: The supports for the reaction load must be located in such a position that they will not affect the slab movements or cause movement of the dial beam support points.

Computation of Volumetric Displacement.—It is assumed the shape of the contours of equal vertical slab movement will be elliptical. A curve of vertical displacement versus distance from the center of the loaded area is drawn for each of the two lines of dial extensometer measurements. The maximum vertical displacement is divided into a convenient number of intervals, and a mechanical integration is made by multiplying the average radius for each increment of deflection from Beam No. 1 times that of Beam No. 2 times p times the deflection interval, *i.e.*, volume of one deflection interval equal π (depth) (Radius 1) (Radius 2). The summation of volumes computed for all deflection intervals is the total volume displaced.

Computation of Volumetric k.—The volume displaced is plotted versus load applied. Volumetric k is taken as the slope of the load vs volume curve and is expressed in pounds per cubic inch.

10.2.2 Development of Test

The Volumetric k Test was developed by engineers of the Rigid Pavement Laboratory, Ohio River Division Laboratories, U. S. Army Engineer Division, Ohio River, Corps of Engineers. The first tests were conducted in 1942 to evaluate size of plates to be used in the evaluation of subgrade reaction. A 20- by 20-ft square isolated slab 10-in. thick was loaded at its center by a 30-in. diameter plate. The volume displacement of the slab under each load was calculated from the slab deflections at the center and along its axis of symmetry. The results of tests on the slab were compared with results of tests on various sizes of plates on the subbase. This led to the adoption of a 30-in. diameter plate by the Corps of Engineers. These tests are reported in "Field Bearing Tests Applied to Pavement Designs," by Robert R. Philippe, Symposium on Load Tests of Bearing Capacity of Soils, Special Technical Publication No. 79, ASTM, 1948, pp. 65-70.

Since the early evaluation tests the Volumetric k Test has been used to evaluate subgrade reaction under test pavements and operational pavements without significant distress to the concrete.

The procedure used by the Rigid Pavement Laboratory is to apply load with the test vehicle and measure total volume displaced. This is done at frequent intervals on test tracks with resulting k-values compared to subgrade or subbase plate bearing test conducted before and after traffic.

The Volumetric k Test has been used on a wide variety of applications by the Rigid Pavement Laboratory. These include plain concrete pavements 6 in. to 32 in. in thickness, reinforced concrete pavements, and prestressed pavements; also, rigid and non-rigid overlay of rigid pavements. In laboratory model tests the volumetric displacement procedure has been used on 1-in. thick prestressed concrete on 4-ft thick natural subgrade materials.

10.3 FROST DEPTH DETERMINATION

A device was developed at the Road Test by which determinations of "depth of frost" could be made without disturbing the pavement. The system was based upon the fact that the electrical resistance of a soil-water system changes rapidly upon freezing. Pairs of electrodes buried in the soil at 1-in. intervals of depth were connected to leads that were brought to the surface. Measurements of the resistance across these electrodes indicated the depth to which the soil-water system had frozen. The system, described in a paper "Frost Depth Determinations by Electrical Resistance Measurements," *Highway Research Abstracts*, Volume 27, April 1957, was used extensively at the Road Test.

Eighty installations were made in traffic and non-traffic test sections. Through their use a record was maintained of the depth of frost penetration and the depth and rate of thawing of a frozen layer either in the upward or downward direction.

A complete record of the frost depth determinations for the Road Test is available in Data Systems 3140 and 3240.

10.4 DRIVER BEHAVIOR STUDIES

Studies suggested by the Subcommittee on Human Reactions of the Special Studies Panel were conducted by the U. S. Army Personnel Research Office, Office of the Chief of Research and Development, Department of the Army.

The purpose of this study was to study the alertness of personnel engaged in a fatiguing and monotonous driving task. Alertness was measured on an hour-by-hour basis for one group of drivers to observe the level and slope of performance. Another group of drivers were administered a battery of psychological tests and measures in an attempt to predict individual differences in alertness.

The test instrument used to measure alertness was designed by members of the AASHO staff from a concept furnished by psychologists of the Army Personnel Research Office.

The results of these studies are available as Technical Research Notes 118 and 119 of the Army Personnel Research Office.

10.5 DYNAMIC TESTING—SHELL ROAD VIBRATION MACHINE

Measurements with the Shell Oil Company Road Vibration Machine were made on selected test sections on the Road Test during the period November 1958 to May 1959.

The tests were conducted on a regular schedule during traffic operations. The eight sections selected for the study included four sections subjected to 18-kip single axle loads (Loop 3) and four replicate sections in the no-traffic loop (Loop 1). Design thicknesses for these sections were 3-0-8, 5-0-8, 3-6-8 and 5-6-8 (surface, base and subbase thickness in inches).

The primary objectives of the study were: to investigate the seasonal variation in stiffness of pavements; to attempt to relate pavement stiffness to pavement performance; to measure properties of the layered construction by nondestructive velocity testing; to compare measured stiffness with stiffness calculated from the properties of the layers; and to compare the properties of sections subjected to traffic with sections without traffic.

The analyses of the velocity and stiffness data are described in the paper "Dynamic Testing at the AASHO Road Test," Bulletin WRP7-59, Shell Oil Company. Some of the conclusions from this study are as follows:

1. The stiffness of the asphalt pavements is greatly influenced by seasonal changes.
2. Seasonal recovery of the pavement structure is indicated, but the degree of recovery requires further investigation.
3. The stiffness of pavement sections with bituminous-treated bases is very high.

The basic field data are available in AASHO Road Test Data Systems 9170 and 9171.

10.6 DYNAMIC TESTING—U. S. ARMY CORPS OF ENGINEERS

During the special studies program in the spring of 1961, a test program was conducted using a dynamic testing device developed by the Waterways Experiment Station, U. S. Army Corps of Engineers.

Personnel from the Waterways Experiment Station, with the assistance of the project staff, made the field tests and will review and analyze the data. It is presumed that the data collected on the Road Test will be included in a report of the development and use of this equipment. Further information will be available upon request to the Director of the Waterways Experiment Station, Vicksburg, Miss.

10.7 SKID STUDIES

Studies of the resistance to skidding of wet and dry flexible and rigid type pavements of known design and traffic treatment were conducted at the Road Test during the traffic phase of the main test.

The uniformly constructed test facility and the controlled test traffic operations offered a unique opportunity to observe the effects of the axle load and arrangement, load applications, pavement design and skid test vehicle speed on the skid resistance of the pavement surface.

The experiment included 80 test sections of different design. Sixteen sections, eight in each traffic lane, were selected from each of the five test loops. Six series of tests are reported in this section. The first was completed prior to any traffic operations and the sixth after more than 1,000,000 loaded axle applications.

Details of the experiment and a summary of the test data are included in Appendix A.

The findings of this experiment are as follows:

1. The most significant change in the skid resistance properties of the pavement sections observed in the Road Test experiment was the reduction in the coefficients of friction under wet pavement conditions resulting from an increase in the number of axle applications in this experiment.

The coefficients of friction prior to the start of the test traffic averaged 0.72 at 30 mph for the flexible pavements and 0.63 at 30 mph for the rigid pavements. After two years of test traffic and 1,101,000 axle applications, the coefficients of friction were reduced to an average of 0.44 for the flexible pavements and 0.42 for the rigid pavements.

Thus, the 1,101,000 axle applications resulted in a 39 percent reduction in the coefficient of friction for the flexible pavements and a 33 percent reduction in the coefficient of friction for the rigid pavements. It should be noted, however, that the original coefficients for the flexible sections were higher than those for the rigid pavements.

2. For the light truck traffic with 2,000-lb axle loads, the reduction in the coefficients of

friction for the 1,101,000 axle applications was considerably lower than for the heavy truck traffic.

For the flexible pavements the coefficients of friction at 30 mph were reduced from 0.76 to 0.65 and for the rigid pavements from 0.69 to 0.59 or a reduction of 14.5 percent for both pavement types.

3. The effect of axle loads ranging from 2 kips to 48 kips on the skid resistance of the Road Test pavements is shown most effectively in the results of skid test measurements made during the summer of 1960 after 851,000 axle applications. (See summary finding No. 5.)

The highest coefficients of friction in this series of tests were obtained on Loop 2 on which vehicles with 2-kip and 6-kip axle loads operated. For the flexible pavements on Loop 2, the skid test coefficients at 30 mph were 0.59 for the 2-kip and 0.47 for the 6-kip axle load test sections; for the rigid pavements the corresponding coefficients were 0.52 and 0.43.

The lowest coefficients of friction in this series of tests were obtained on pavements in Loop 6 on which vehicles with 30-kip single and 48-kip tandem axle loads operated. For the flexible pavements on Loop 6, the coefficients of friction at 30 mph averaged 0.34 for the 30-kip single and 0.33 for the 48-kip tandem axle load test sections; for the rigid pavements the corresponding coefficients were 0.37 and 0.35.

For the pavements on which vehicles operated with various incremental increases in axle load to provide total axle loads of 2, 12, 18, 22.4, and 30 kips for the single axles and of 6, 24, 32, 40, and 48 kips for the tandem axles, the coefficients of friction decreased at a fairly uniform rate from the highest values for the 2- and 6-kip axle load sections to the lowest values for the 30-kip single and 48-kip tandem axle load sections.

4. A significant reduction in the coefficients of friction was noted as the speed of the skid test vehicle was increased on all of the test sections during all stages of the Road Test. For the flexible pavement test sections, the mean coefficient of friction for all tests conducted at 10 mph was 0.68 and at 50 mph it was 0.42. For the rigid pavement test sections the corresponding values were 0.69 and 0.38. Pavement roughness or serviceability, in the range from good to fair, did not appreciably influence this relationship.

5. A marked change in the coefficients of friction due to seasonal and/or weathering effects was noted in the results of the skid tests. In general, the coefficients of friction measured during early spring of 1960 were 20 to 35 percent higher than the friction values measured in tests conducted in the summer of 1960.

6. The coefficients of friction for pavements of different structural thicknesses were reasonably uniform over the range of pavement thickness selected for the skid tests. It should be noted, however, that in this respect the tests were limited to the thicker pavement sections which were selected to provide reasonable assurance that the composition of the pavement surface of these sections would not be changed by maintenance operations such as overlays required by the test traffic during the 2-yr period in which the Road Test was in operation.

Appendix A

SKID TESTS*

THE EXPERIMENT

Five controlled variables were selected for evaluation in this experiment. They were pavement design, axle load and arrangement, speed of the skid test equipment, pavement surface condition and load applications. An outline of the first three variables is given in Tables 1-A and 2-A.

In addition to the main experiment a partial study of the effect of the condition of the pavement surface (wet or dry) was included. All of the sections in Tables 1-A and 2-A were tested in the wet surface study, but only selected sections were included in the dry surface study.

Design variables for the rigid pavement sections included two levels of surfacing and subbase thickness for each axle load and arrangement. To insure a reasonably balanced experiment for the duration of the Road Test the higher levels of design for each axle load were selected for study. The possible effect of the joint spacing was observed by the inclusion of both reinforced and plain concrete sections. Transverse contraction joints, formed by sawing, were spaced at 40 ft in reinforced sections and 15 ft in non-reinforced sections.

Similarly the design variables for the flexible pavement sections included two highest levels of surfacing, base and subbase thickness for each axle load and arrangement.

The possible effect of the speed of the skid test vehicle was investigated in a partial study. All sections in Tables 1-A and 2-A were included in the main experiment at 30 mph and in addition, those underlined were tested at 10 and 50 mph.

The most pronounced effect on the skid resistance coefficients was anticipated to be as a result of the accumulation of load applications. To determine this possible effect, the six series of tests were scheduled at fairly regular intervals during the test traffic phase of the Road Test. The date of each series and the accumulated axle applications are given in Table 3-A.

A total of six single and four tandem axle loads were selected. The pavement designs for the single axle loads of 2, 6, 12, 18, 22.4, and 30 kips and for the tandem axle loads of 24, 32, 40, and 48 kips are given in Tables 1-A and 2-A.

Other variables in the test could be classified as uncontrolled. Of these the most important appeared to be the environmental conditions. Among the environmental conditions measured independently were the air temperature, pavement temperature and the rainfall preceding the test series. Air temperature varied between 36 F and 94 F, pavement temperature between 37 F and 123 F, and the two-week rainfall prior to the test series varied from 0.36 to 1.77 in.

The dry surface studies were run in three series of tests. The sections selected for this study were those chosen for the special speed study and were in traffic lanes carrying vehicles with axle loads of 22.4-kip single and 40-kip tandem.

The mix designs, method of placement and finishing techniques of the surfacing courses, either asphaltic or portland cement concrete, were essentially the same throughout the Road Test. Figure 1-A is a typical example of the surface texture of the two pavement types at the start of test traffic.

The General Motors Corporation skid trailer (Fig. 2-A) was used in all test series of the skid study. The skid resistance is described by the coefficient of friction and is computed from the known characteristics of the testing equipment and the measured force required to pull the trailer with the wheels locked.*

TEST RESULTS

Despite rigid inspection of the finishing operations for both types of pavements, substantial differences were noted in the coefficients of friction before the start of test traffic. Values of the coefficient of friction for the flexible pavement sections ranged from 0.76 for those designed for the 2- and 6-kip single axle loads to 0.67 for those designed for the 40-kip tandem axle loads. For the rigid pavement sections the range was from 0.70 for those designed for the 2- and 6-kip single axle

* Adapted from a paper given at the 40th Annual Meeting of the Highway Research Board and published in HRB Special Report 66.

* Skeels, P. C., "Measurements of Pavement Skidding Resistance by Means of a Simple Two-Wheel Trailer." HRB Bull. 186, pp. 33-45 (1958).

TABLE 1–A

OUTLINE OF SKID STUDY [1], FLEXIBLE PAVEMENTS

Axle Load [2] (kips)	Sub-base (in.)	2-Inch Surface		3-Inch Surface		4-Inch Surface			5-Inch Surface			6-Inch Surface		
		3-In. Base	6-In. Base	3-In. Base	6-In. Base	3-In. Base	6-In. Base	9-In. Base	3-In. Base	6-In. Base	9-In. Base	3-In. Base	6-In. Base	9-In. Base
2KS	0	X̲	X	X	X̲	—	—	—	—	—	—	—	—	—
	4	X	X̲	X̲	X	—	—	—	—	—	—	—	—	—
6KS	0	X̲	X	X	X̲	—	—	—	—	—	—	—	—	—
	4	X	X̲	X̲	X	—	—	—	—	—	—	—	—	—
12KS	4	—	—	X̲	X	X̲	X̲	—	—	—	—	—	—	—
	8	—	—	X	X̲	X̲	X	—	—	—	—	—	—	—
24KT	4	—	—	X̲	X	X	X̲	—	—	—	—	—	—	—
	8	—	—	X	X̲	X̲	X	—	—	—	—	—	—	—
18KS	8	—	—	—	—	X̲	X	—	X	X̲	—	—	—	—
	12	—	—	—	—	X	X̲	—	X̲	X	—	—	—	—
32KT	8	—	—	—	—	X̲	X	—	X	X̲	—	—	—	—
	12	—	—	—	—	X	X̲	—	X̲	X	—	—	—	—
22.4KS	8	—	—	—	—	—	X̲	X	—	X	X̲	—	—	—
	12	—	—	—	—	—	X	X̲	—	X̲	X	—	—	—
40KT	8	—	—	—	—	—	X̲	X	—	X	X̲	—	—	—
	12	—	—	—	—	—	X	X̲	—	X̲	X	—	—	—
30KS	12	—	—	—	—	—	—	—	—	X̲	X	—	X	X̲
	16	—	—	—	—	—	—	—	—	X	X̲	—	X̲	X
48KT	12	—	—	—	—	—	—	—	—	X̲	X	—	X	X̲
	16	—	—	—	—	—	—	—	—	X	X	—	X	X

[1] All sections tested at 30 mph; underlined sections at 10, 30, and 50 mph.
[2] S = single axle; T = tandem axle.

TABLE 2–A

OUTLINE OF SKID STUDY [1], RIGID PAVEMENTS

Axle Load [2] (kips)	Sub-base (in.)	3.5-In. Surface		5.0-In. Surface		6.5-In. Surface		8.0-In. Surface		9.5-In. Surface		11.0-In. Surface	
		Non-Reinf.	Reinf.	Non-Reinf.	Reinf.	Non-Reinf.	Reinf.	Non-Reinf.	Reinf.	Non-Reinf.	Reinf.	Non-Reinf.	Reinf.
2KS	3	X̲	X	X	X̲	—	—	—	—	—	—	—	—
	6	X	X̲	X̲	X	—	—	—	—	—	—	—	—
6KS	3	X̲	X	X	X̲	—	—	—	—	—	—	—	—
	6	X	X̲	X̲	X	—	—	—	—	—	—	—	—
12KS	3	—	—	X	X̲	X̲	X	—	—	—	—	—	—
	6	—	—	X̲	X	X	X̲	—	—	—	—	—	—
24KT	3	—	—	X	X̲	X̲	X	—	—	—	—	—	—
	6	—	—	X̲	X	X	X̲	—	—	—	—	—	—
18KS	3	—	—	—	—	X	X̲	X̲	X	—	—	—	—
	6	—	—	—	—	X̲	X	X	X̲	—	—	—	—
32KT	3	—	—	—	—	X	X̲	X̲	X	—	—	—	—
	6	—	—	—	—	X̲	X	X	X̲	—	—	—	—
22.4KS	3	—	—	—	—	—	—	X	X̲	X̲	X	—	—
	6	—	—	—	—	—	—	X̲	X	X	X̲	—	—
40KT	3	—	—	—	—	—	—	X	X̲	X̲	X	—	—
	6	—	—	—	—	—	—	X̲	X	X̲	X̲	—	—
30KS	3	—	—	—	—	—	—	—	—	X	X̲	X̲	X
	6	—	—	—	—	—	—	—	—	X̲	X	X	X̲
48KT	3	—	—	—	—	—	—	—	—	X	X̲	X̲	X
	6	—	—	—	—	—	—	—	—	X̲	X	X	X̲

[1] All sections tested at 30 mph; underlined sections at 10, 30, and 50 mph.
[2] S = single axle; T = tandem axle.

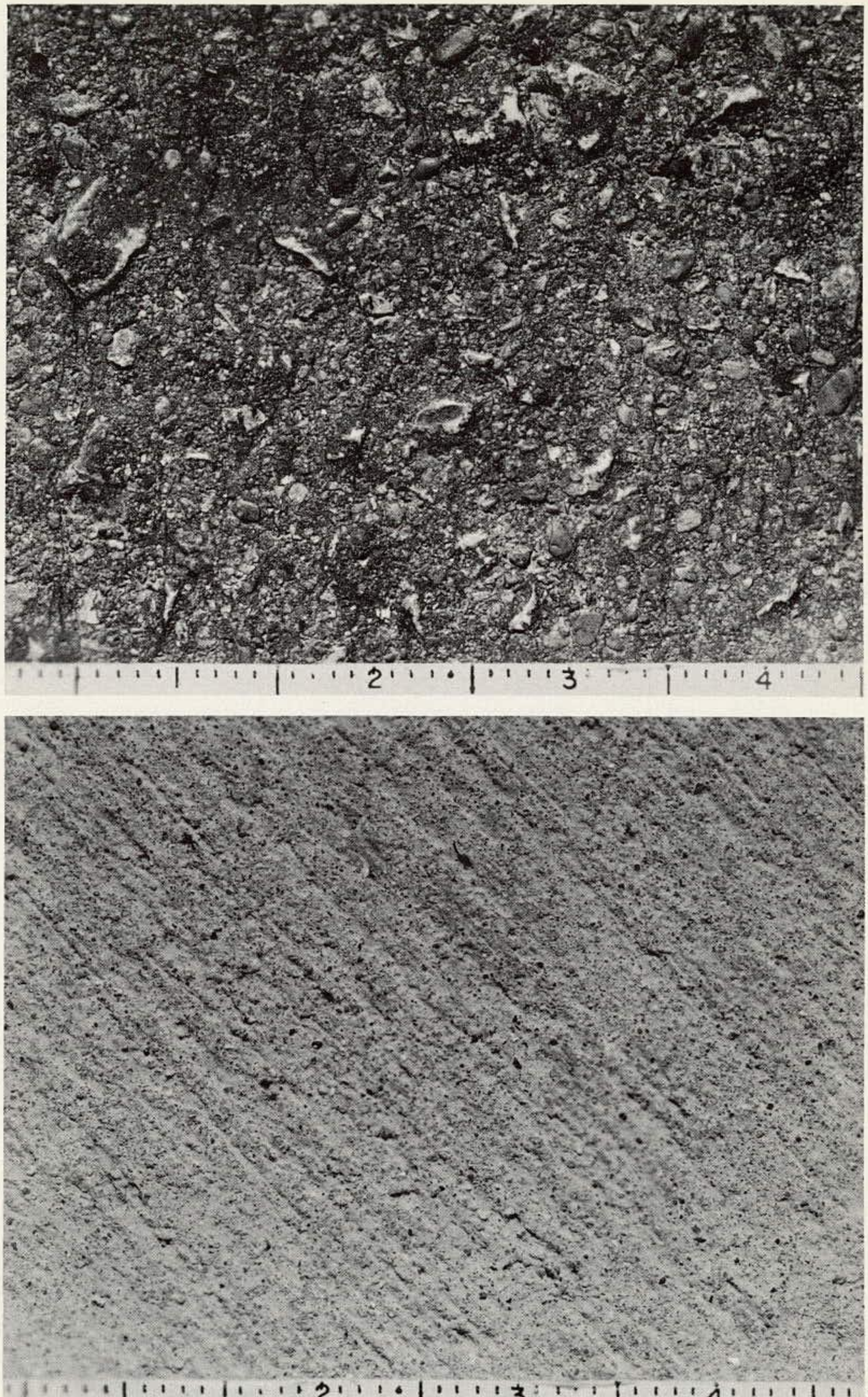


Figure 1–A. Surface texture of asphaltic concrete (top), and of portland cement concrete (bottom).

loads to 0.60 for those designed for the 22.4- and 30-kip single axle loads. Thus the initial coefficients of friction for both rigid and flexible pavements were higher for those sections designed for the 2- and 6-kip single axle load sections. An explanation of this might be in the lighter roller weights used on the thinner designs for the flexible sections and in the stiffness of the mix required for the rigid sections.

Standard deviations of the mean coefficients of friction for the first series were 0.030 for all flexible pavement sections and 0.050 for all rigid pavement sections. The initial variations between the pavements for each load were observed throughout the testing period. However, variations within sections for the same load were reduced considerably with each series of tests. Standard deviations for the last series of tests were 0.020 for all flexible pavement sections and 0.025 for all rigid pavement sections. The test indicated the replication error of the testing equipment was within 0.020 units.

Effect of Pavement Design

The design variable was investigated because of the belief that the increasing roughness of the thinner pavement designs might introduce an increase in the coefficients of friction. With reference to Tables 1-A and 2-A, the range of design thicknesses incorporated within this experiment is from 5 to 31 in. for the flexible pavement sections and from 6.5 to 17.0 in. for the rigid pavement sections.

Table 4-A allows comparison of mean coefficients of friction for wet pavements across the various design levels. Comparisons can be made to examine the effect of surfacing, base and subbase thicknesses and the effect of joint spacing in the portland cement concrete on the coefficients of friction. Results from the six test series were combined to develop these means. For example, the greatest difference between the coefficients for the effect of the concrete reinforcement or joint spacing is 0.01 units. This difference, occurring in all loops, is well within the replication error of the experiment and cannot be considered significant.

Similar comparisons of the effect of the design variables on the coefficients of friction indicated quite clearly that, within the limits of the study, pavement design has no significant effect on the resistance to skid.

Effect of Axle Load and Axle Arrangement

The effect of axle load and axle arrangement (single or tandem) on the pavement surface wear is shown in Figures 3-A and 4-A. The reduction of the coefficient of friction is plotted against wheel load with the reduction plotted upwards indicating a decrease in the coefficient.

Each plotted point is the mean of 48 tests, six series on the eight sections for each load. There appears to be a clear distinction between

TABLE 3–A
HISTORY OF AXLE APPLICATIONS

Series	Time	Accum. Axle Applications
1	Fall 1958	0
2	Spring 1959	108,000
3	Summer 1959	232,000
4	Spring 1960	586,000
5	Summer 1960	851,000
6	Fall 1960	1,101,000

Figure 2-A. General Motors Corporation skid trailer.

the single axle loads and tandem axle loads, the tandem axle loads causing a greater reduction in the coefficients of friction.

There is little indication of any over-all trend that would suggest a reduction of the coefficient of friction due to an increase in load in the lighter axle loadings. However, there is a slight indication that this may be true for the heavier axle loadings. For both the flexible and rigid pavements a greater reduction of the coefficient of friction was experienced with the 30-kip single and 48-kip tandem axle loads than for the 22.4-kip single and 40-kip tandem axle loads.

The high reduction in the coefficient of friction for test sections designed for the 3-kip wheel load suggests a possible effect of the front axle. The only axle loads counted as axle load applications were those with the selected load. To keep the rate of selected load applications the same for pickup trucks and tractor-trailer combinations, it was necessary to have double the number of vehicle trips carrying the 3-kip wheel load. The greater number of un-

TABLE 4–A

MEAN COEFFICIENT OF FRICTION FOR VARIOUS COMBINATIONS[1] OF PAVEMENT DESIGN AND LOAD[2]

Design Feature	Mean Coefficient of Friction				
	2-6KS	12KS 24KT	18KS 32KT	22.4KS 40KT	30KS 48KT
Rigid pavement:					
Reinforced	0.57	0.50	0.52	0.50	0.46
Nonreinforced	0.56	0.49	0.53	0.50	0.47
Surface thickness:					
Second level	0.57	0.49	0.53	0.50	0.46
Third level	0.57	0.51	0.52	0.50	0.47
Subbase thickness:					
3 in.	0.57	0.51	0.53	0.49	0.47
6 in.	0.57	0.49	0.52	0.50	0.46
Flexible pavement:					
Surface thickness:					
First level	0.62	0.56	0.56	0.54	0.50
Second level	0.62	0.53	0.55	0.54	0.47
Base thickness:					
First level	0.62	0.53	0.56	0.54	0.49
Second level	0.62	0.56	0.56	0.55	0.48

[1] Flexible subbase values not shown—incomplete study.
[2] All values are mean of six series of tests except those for the 12-kip single and 24-kip tandem axle loads, which were mean of four series.

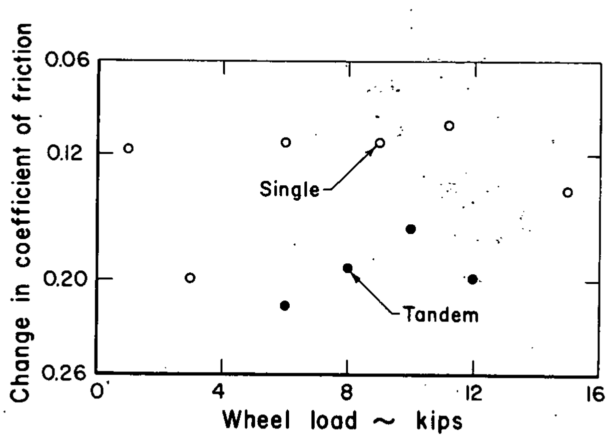


Figure 3-A. Influence of wheel load, rigid pavement.

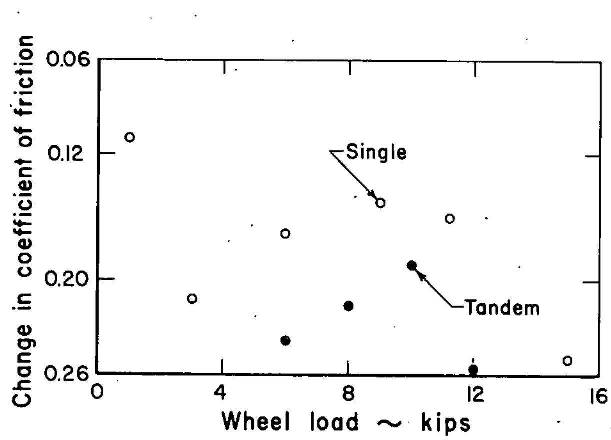


Figure 4-A. Influence of wheel load, flexible pavement.

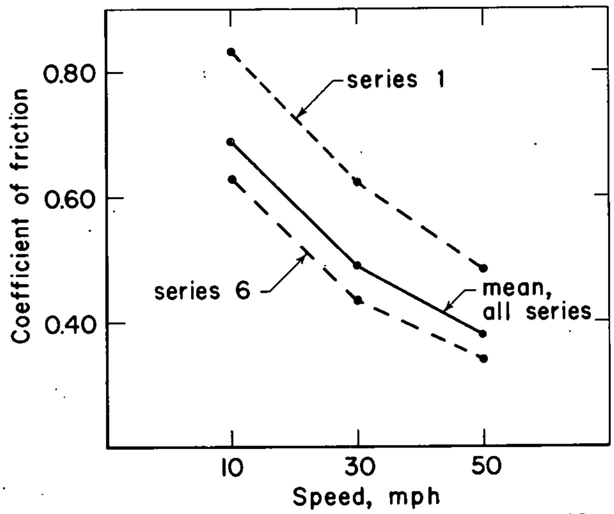


Figure 5-A. Influence of test vehicle speed, rigid pavement.

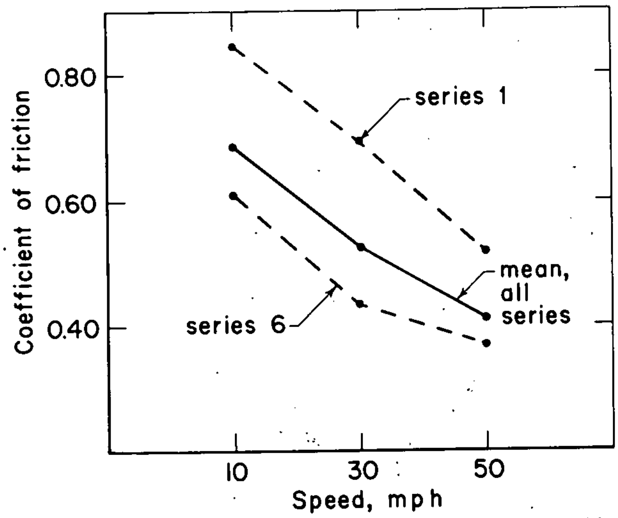


Figure 6-A. Influence of test vehicle speed, flexible pavement.

counted steering axles may account for the high reduction in the coefficient of friction.

Effect of Test Vehicle Speed

Figures 5-A and 6-A show the effect of the speed of the testing vehicle on the coefficients of friction measured on wet pavements. Three curves are plotted in each figure showing, along with the mean relationships for all series of tests, the relationships when the test sections were newly constructed and at the end of the traffic testing period.

For each test series, the results of the measurements on the 2-kip and 6-kip single axle load sections were deleted because of incomplete data. Thus a point on the curve is the mean of 32 tests, four tests for each of the remaining loads. A point on the curve for all test series is the mean of 192 tests, four tests for each of the remaining loads for each series.

For all tests on wet pavements, the measured coefficient of friction was substantially reduced with an increase in speed of 20 mph. The relationship appears to be curvilinear within the range of the test data. For tests on dry pavements the speed of the testing equipment had very little effect.

It has been shown in tests that for an excellent pavement the coefficient of friction was only slightly affected by the speed of the testing equipment whereas for a poor pavement there was a substantial reduction. The curves for test series one and six (Figs. 5-A and 6-A) clearly indicate that the condition of the pavements on the Road Test had little effect on the influence of the speed of the test vehicle.

Effect of Pavement Surface Condition

Figures 7-A and 8-A for the rigid and flexible pavement surfaces, respectively, show the effect of pavement surface conditions (wet or dry) on the coefficients of friction. Each point is the mean of four coefficients of friction of four test sections.

Dry tests show to a marked degree an increase in the coefficient for both pavement types. The over-all trend of the dry surface coefficients of friction is a decrease with the increasing load applications. Differences between axle loads and axle arrangements do not appear to influence the coefficients within the range of the tests. Also the seasonal variations do not appear to have any significant effect on the dry surface condition coefficients.

Effect of Axle Load Applications

With a range of loaded axle applications from 0 to 1,100,000 for all loads, the influence of the number of applications on the coefficients of friction was expected to be the most significant finding of this experiment.

As mentioned previously (Table 3-A), the six test series were conducted as nearly as possible at regular intervals throughout the test traffic phase of the Road Test.

A typical set of data representing the change in coefficients of friction at 30 mph with increasing load applications for the 22.4-kip single axle load is shown in Figure 9-A.

The over-all trend of the data is a decrease in the coefficient of friction with an increase in axle applications. However, the coefficients show an increase for two periods: from the summer 1959 test series to the spring 1960 test

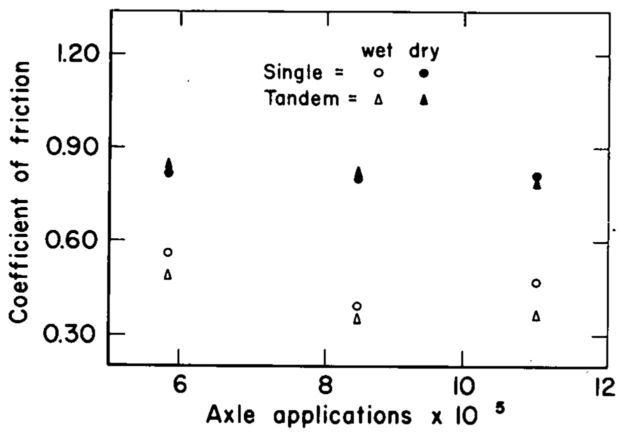


Figure 7-A. Influence of pavement surface condition, rigid pavement.

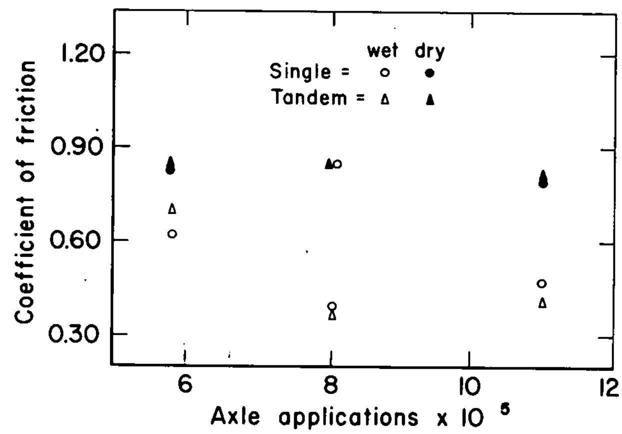


Figure 8-A. Influence of pavement surface condition, flexible pavement.

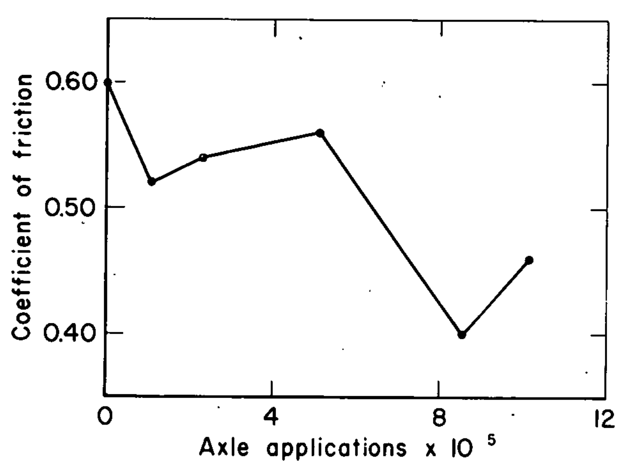


Figure 9-A. Influence of axle applications.

TABLE 5-A

Recorded Weather Phenomena

Series	Rainfall, Two Weeks Preceding (in.)	Temperature Range (°F)		
		Air	Rigid Pvt.	Flexible Pvt.
1	1.22	80–47	84–55	73–91
2	1.77	86–45	102–45	107–46
3	1.63	94–73	113–78	123–84
4	0.53	63–36	63–37	65–37
5	0.36	88–65	119–80	117–81
6	0.79	60–37	62–40	67–39

series, and from the summer 1960 to the fall 1960 test series.

The first increase may be attributed to the influence of the freeze and thaw cycles and the scouring of the pavement surface by heavy rainfall immediately prior to the tests. The second increase could be associated with rainfall before the latter test series which reduced the accumulation of dust and oil slicks on the pavement surface. Furthermore, the heavy rainfall prior to the summer 1959 tests may have reduced the dust and oil slick on the pavement surface resulting in a higher coefficient of friction than would normally be expected. The scouring effect of heavy rainfall would appear, therefore, to have a significant effect on the coefficients of friction. Table 5-A shows the rainfall accumulation for the two-week period preceding each test series.

Other weather phenomena recorded during the test series are also shown in Table 5-A. No apparent effect of these phenomena was observed, but further investigation of possible interactions may show some influence on the coefficients of friction.

DETAILS OF EXPERIMENT

Measurement of Coefficient of Friction

The coefficient of friction is defined in this report as the ratio of the horizontal force required to pull the trailer at a constant speed with the wheels locked to the vertical reaction at the wheels. The vertical reaction is determined by subtracting from the static weight on the trailer wheels the force exerted by a couple produced by the force in the draw bar and the wheel friction. Thus, by measuring the draw bar force the coefficient of friction may be determined. The General Motors skid trailer is designed to measure this force.

TABLE 6–A

SUMMARY OF GRADATION TESTS ON PORTLAND CEMENT CONCRETE AGGREGATES

Sieve Size	Gradation Formula Tolerances	Mean Percent of Material Passing	Standard Deviation
(a) COARSE AGGREGATE SIZE A (170 TESTS)			
2½ in.	100	100	—
2 in.	90–100	96.3	3.45
1½ in.	62±7	63.5	6.11
1 in.	10±5	10.6	3.18
½ in.	0–5	3.8	2.14
(b) COARSE AGGREGATE SIZE B (171 TESTS)			
1½ in.	100	100	—
1 in.	90–100	94.1	1.30
½ in.	38±5	37.9	1.65
No. 4	0–10	1.5	0.78
(c) SAND (80 TESTS)			
⅜ in.	100	100	—
No. 4	95–100	99.0	0.97
No. 8	85±5	84.1	1.55
No. 16	67±4	67.0	1.83
No. 30	46±4	45.4	1.51
No. 50	13±3	12.3	0.73
No. 100	3±2	2.7	0.46

Materials Specification and Finishing Techniques

Portland Cement Concrete.—Two coarse aggregates and one sand were blended together for the portland cement concrete. Both the coarse aggregate and the sand were obtained near the project site. The aggregate was predominantly dolomite, and the sand was mostly siliceous.

The two coarse aggregates had maximum sizes of 2½ in. and 1½ in. and the sand had a finenesss modulus of 2.90. The sieve analysis for the coarse aggregates and sand are given in Table 6-A, and the lithological analysis for the coarse aggregates is given in Table 7-A. Type 1 portland cement was supplied by one manufacturer from one continuous grinding and burning operation.

The design characteristics of the portland cement concrete are shown in Table 8-A. Mean 14-day compressive strengths for concrete containing the 2½-in. and 1½-in. maximum size aggregates were 3,966 psi and 4,004 psi, respectively. Mean 14-day flexural strengths for concrete containing the 2½-in. and 1½-in. maximum size aggregates are 636 psi and 668 psi—the means of 394 and 67 tests, respectively.

TABLE 7–A

LITHOLOGICAL ANALYSIS OF PORTLAND CEMENT CONCRETE COARSE AGGREGATES

Rock Type	Percent Passing, by Weight					
	2–1½	1½–1	1–¾	¾–½	½–⅜	⅜–No. 4
Dolomite	38	47	38	64	59	59
Argillaceous limestone	28	27	23	9	12	14
Soft sandstone	15	12	11	8	11	11
Hard sandstone	0	5	6	4	5	3
Chert	13	4	18	8	7	7
Diabase	4	0	0	2	2	2
Granite	2	2	2	3	3	4
Quartz	0	3	2	2	1	0

The portland cement concrete was finished by the non-vibratory method. After the concrete had been deposited and spread between the forms, it was accurately struck off, screeded and consolidated with at least two passes of a non-vibrating finishing machine. It was further smoothed and consolidated by a mechanical longitudinal float. The floating operation was continued until the surface of the concrete was smooth and at the proper crown and grade.

The surface was checked with a 10-ft straightedge; and when most of the water sheen had disappeared, it was belted with one application of a mechanical belt. This was followed by edging, and final finish was obtained with two passes of a double thickness burlap drag.

Immediately after the finished concrete had attained a sufficient set, it was covered with two layers of burlap. The burlap blanket was saturated with water and kept wet until moved. The morning following the placement of the concrete, the forms and burlap were removed and the surface and edges of the pavement

TABLE 8–A

DESIGN CHARACTERISTICS, PORTLAND CEMENT CONCRETE

Surf. Thick. (in.)	Cement Content (bags/cu yd)	Water-Cement Ratio (gal/bag)	Sand Volume (% tot. aggr. vol.)	Air Content (%)	Slump (in.)	Max. Aggr. Size (in.)
5 and greater	6.0	4.8	32.1	3–6	1.5–2.5	2.5
2.5–3.5	6.0	4.9	34.1	3–6	1.5–2.5	1.5

TABLE 9-A

SUMMARY OF EXTRACTION TEST RESULTS [1]

Sieve	Mix Design	Mean Value	Standard Deviation
¾ in.	100	100	—
½ in.	90±5	92	2.43
⅜ in.	80±5	81	3.17
No. 4	64±5	63	4.06
No. 10	45±4	46	2.99
No. 20	31±4	34	1.66
No. 40	20±4	22	2.06
No. 80	11±3	13	1.07
No. 200	5±1	5.9	1.16
Asphalt [2] (%)	5.4±0.3	5.2	0.18

[1] Ninety-six tests on surface course mixture.
[2] Percent asphalt by total weight of mix. Control tests have shown that the extraction tests underestimated asphalt by 0.1 to 0.2 percentage points.

were covered with a layer of clean straw. The straw was then saturated with water, attaining a wet thickness of approximately 8 in. and was kept wet for the first three days. It was thoroughly wet down on the morning of the fourth day and remained in place until after test beams indicated that the concrete had attained a flexural strength of at least 500 psi.

Asphaltic Concrete, Surface Course.—The coarse aggregate was predominantly crushed dolomitic limestone from near the project site. The maximum size for the surfacing course was ¾ in. Two sands, coarse and fine, were blended together for the fine aggregate to a specified fineness modulus of 2.35. The grain size analysis and percent of asphalt for the 96 extraction tests on surface course material are given in Table 9-A. Other characteristics of the surface material are: Marshall stability, 2,000 lb; flow, 0.11 in.; percent voids by volume, 3.6; and percent voids filled with asphalt, 77.9.

Bituminous construction was performed in lane widths. Two spreading and finishing machines were used, one for each lane. While construction operations were being performed in one lane, the other machine was being positioned in the opposite lane so that the crew could move back and immediately start spreading in that lane. Material on hand sufficient to insure a continuous spreading operation throughout a test section was maintained at all times. Delays in operation were confined to transition areas, except on rare occasions due to equipment failure.

Compaction of each layer of bituminous mixture required the use of a 3-wheel roller followed by a self-propelled pneumatic-tired roller, with final rolling by a tandem roller. Pneumatic-tired rollers were not being used very extensively for compacting bituminous concrete but experimental work indicated that the attained level of density more nearly corresponded to that produced by traffic on existing highways. The other requirements pertaining to the time of rolling and the speed and procedure for compacting the bituminous concrete courses were in line with normal construction practice.

The thickness of the subbase plus base was used as a guide in selecting the proper set of rollers for each structural section. At any time that there was an indication that a section was being damaged or might be damaged with the set of rollers being used, that set was immediately removed and replaced with the next lighter set of rollers or the number of passes of the rollers was reduced. Roller weights and mat temperatures used are shown in Table 10-A.

TABLE 10-A

ROLLING WEIGHTS AND TEMPERATURES FOR BITUMINOUS CONCRETE CONSTRUCTION

Roller Set	Roller Weights (lb/in. width)			Section Thickness [2] (in.)
	Three Wheel [1]	Pneumatic Tired	Tandem	
Heavy	300	300	250	15 (also all 9″ base sections)
Intermediate	214	250	190	8 to 15
Light	180	200	120	8 or less

Mat Rolling Temperatures (°F)		
Three Wheel	Pneumatic Tired	Tandem
250–275	190–220	— [3]

[1] Based on 9-in. tire tread, inflation pressure 75 psi.
[2] Subbase plus base.
[3] While mat was still workable but had cooled sufficiently to prevent shoving.

Usually one pass of the 3-wheel roller followed by eight passes of the pneumatic-tired roller was sufficient to obtain the required density on the layer being compacted. The tandem roller was considered only as a finish roller to remove the roller marks of the 3-wheel and pneumatic-tired rollers, and sufficient number of passes were made over a layer to accomplish this. Experimental work indicated that very little, if any, additional increase in density was obtained with the tandem roller.

A steel bristle broom drag was placed behind the spreading and finishing machine for the placement of the surface course to correct any slight tearing that might occur.

Tire Sizes and Pressures

Table 11-A gives details of tire sizes and pressures for each load. Numerous makes of new tires and types of recaps were used in the operation and no attempt has been made to associate any make with a particular axle load. The pressure shown should be considered as nominal cold measurement.

TABLE 11–A

TIRE SIZES AND PRESSURES

Axle Load (kips)	Tandem or Single Axle	Tire Size	Tire Pressure (psi)
2	S	6:70x15	24
6	S	7:00x16	45
12	S	7:50x20	75
24	T	7:50x20	75
18	S	10:00x20	75
32	T	9:00x20	70
22.4	S	11:00x20	75
40	T	11:00x20	75
30	S	12:00x24	80
48	T	12:00x20	80

TABLE 12-A

COEFFICIENT OF FRICTION FOR 2- AND 6-KIP SINGLE AXLE LOADS

Pavement Design[1]	Coefficient of Friction																	
	Series 1			Series 2			Series 3			Series 4			Series 5			Series 6		
	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph
2–3–0		80		87	60	53	83	60	53	85	76	60	70	53	49	77	64	54
2–3–0		79		81	57	46												
2–3–4		77			62			60			72			59			67	
2–3–4		73			55			55			62							
2–6–0		73			62			64			73			60			66	
2–6–0		72			59			56			62			50			57	
2–6–4		83		91	67		86	62	44	88	72	53	75	61	46	49	65	47
2–6–4		82		81	61		75	56	45	76	64	53	54	45	43	64	49	38
3–3–0		70			62			58			73			61			66	
3–3–0		74			53			56			62							
3–3–4		77		89	66	57	82	65	54	87	75	62	75	62	48	78	65	53
3–3–4		77		84	56	42	80	58	48	74	64	58	57	45	39	67	50	42
3–6–0		74		86	62	53	83	64	52	84	74	64	77	57	47	77	62	51
3–6–0		71		83	57	44	77	56	49	75	67	57	58	45	37	64	48	42
3–6–4		73			62			63			74			62			66	
3–6–4		78			49			56			62			48			50	
3.5–3		68		81	61	47	84	58	46	81	64	51	76	54	44	76	60	44
3.5–3		74		79	53	38	68	53	40	75	60	51	62	46	33	64	45	37
3.5–6		68		80	58	46	83	55	43	79	62	49	73	51	42	76	58	44
3.5–6		68		79	49	35	72	53	43	76	65	47	60	42	33	58	41	34
5–3		69			49			57			55			44			56	
5–3		68			46			57			51			40			43	
5–6		72		86	62	47	85	56	46	83	66	53	77	52	46	80	58	47
5–6		70		81	55	39	75	54	41	82	59	50	61	47	38	65	47	38
3.5R–3		72			62			57			66			54			62	
3.5R–3		72			55			53			67			44			47	
3.5R–6		62		82	56	46	81	55	41	79	60	52	74	52	43	76	57	42
3.5R–6		66		81	53	40	75	52	44	84	53	42	60	43	33	66	44	33
5R–3		69		85	58	46	84	57	45	82	64	54	76	52	43	77	59	46
5R–3		71		81	53	37	73	52	39	78	59	50	60	42	33	66	45	38
5R–6		74			59			57			64			54			62	
5R–6		73			53			54			58			57			46	

[1] Surface thickness, base thickness, subbase thickness, all in inches.

TABLE 13–A

COEFFICIENT OF FRICTION FOR 12-KIP SINGLE AND 24-KIP TANDEM

Design[1] Pavement	Coefficient of Friction: Series 1			Series 2			Series 3			Series 4			Series 5			Series 6		
	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph
3–3–4	92	72	55															
3–3–4	90	69	48															
3–3–8		64			55			50										
3–3–8		68			50													
3–6–4		67			60			62			55							
3–6–4		68																
3–6–8	88	70	56	86	63	44	76	53	47	77	50	41	58	48	36	64	46	34
3–6–8	92	70	57	86	50	35	65	46	34	58	43	36						
4–3–4		71			63						52							
4–3–4		70									36							
4–3–8	90	68	53	79	53	44	76	57	47	71	53	40						
4–3–8	88	66	50	77	47	43	74	47	40	66	42	35						
4–6–4	88	70	53	80	61	50	77	56	46	72	52	41	60	49	38	70	49	41
4–6–4	90	71	51	77	65	36	69	46	32	64	42	33						
4–6–8		68			58			54			51			45			48	
4–6–8		70			51			44			44			41			43	
5–3		56			51			51			56			43			48	
5–3		57			40			40			42							
5–6	83	55	53	79	50	42	69	49	38	78	56	38	77	48	36	73	53	37
5–6	71	48	35	66	40	37	58	38	31	78	45	32	67	43	29			
6.5–3	79	56	44	77	55	44	74	53	40	73	57	36	68	49	36	71	47	36
6.5–3	79	62	44	64	40	35	67	45	40	70	43	35	70	45	29	64	43	37
6.5–6		64			49			52			52			48			46	
6.5–6		65			39			36			39			40			36	
5R–3	85	66	51	75	55	38	71	50	39	70	47	37	65	45	32	63	44	36
5R–3	92	70	53	63	46	32	63	44	22	64	34	27	68	40	27			
5R–6		62			49			49			43							
5R–6		64			35			43			33							
6.5R–3		66			52			54			52			47			46	
6.5R–3		69			45			43			38			37			38	
6.5R–6	88	64	47	74	53	40	73	51	40	72	54	35	72	46	34	70	47	37
6.5R–6	85	60	47	62	43	33	55	40	35	69	40	27	60	42	26	59	36	31

[1] Surface thickness, base thickness, subbase thickness, all in inches.

TABLE 14–A

Coefficient of Friction for 18-Kip Single and 32-Kip Tandem

Pavement Design[1]	Coefficient of Friction																	
	Series 1			Series 2			Series 3			Series 4			Series 5			Series 6		
	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph
4–3–8	83	74	50	76	60													
4–3–8	83	69	52	73	53													
4–3–12		70			62			62			69			42			52	
4–3–12		70			51			54			60			33				
4–6–8		72			66			60			64			42			49	
4–6–8		73			49			51			55			36			40	
4–6–12	85	70	59	89	63		76	58	52	83	67	49	62	47	41	62	49	45
4–6–12	87	69	49	83	61		71	51	47	68	60	34	63	36	29	53	42	37
5–3–8		68			68			56			62							
5–3–8		72			62			49			59			34				
5–3–12	81	74	56	77	66		75	63	53	82	68	48						
5–3–12	85	67	53	75	78		70	51	43	64	59	36	47	32	25	54	41	32
5–6–8	83	77	56	65	52		75	62	51	87	64	51	70	49	37	66	51	47
5–6–8	83	71	53	79	53		71	51	41	68	58	32	51	34	25	53	37	31
5–6–12		71			65			58			67			42			52	
5–6–12		68			51			50			62			41			45	
6.5–3		76			57			58			62			42			49	
6.5–3		68			46			51			49							
6.5–6	85	66	50	79	55	46	77	56	48	77	62	46	67	45	35	72	50	43
6.5–6	92	68	53	74	49	43	69	53	45	70	56	42	60	38	30			
8–3	85	61	44	81	51	43	77	56	45	76	58	43	70	47	34	73	54	41
8–3	89	61	45	72	45	32	72	46	38	66	54	34	60	41	30	66	46	33
8–6		59			48			55			62			47			54	
8–6		57			46			49			58			40			46	
6.5R–3	92	69	48	79	49	43	79	54	43	76	58	43	59	40	29	65	48	34
6.5R–3	92	71	50	65	46	36	60	40	39	58	50	31						
6.5R–6		57			51			53		79	59	46		49			55	
6.5R–6		59			47			49		75	57	42						
8R–3		64			52			54		79	58	46		42			50	
8R–3		66			44			47		73	53	42		41			42	
8R–6	89	59	50	76	51	37	73	55	39	67	59	44	58	43	31	71	51	36
8R–6	92	69	53	79	44	37	67	37	35	54	52	32	52	34	24	55	37	30

[1] Surface thickness, base thickness, subbase thickness, all in inches.

TABLE 15–A

COEFFICIENT OF FRICTION FOR 22.4-KIP SINGLE AND 40-KIP TANDEM

Pavement Design[1]	Coefficient of Friction																	
	Series 1			Series 2			Series 3			Series 4			Series 5			Series 6		
	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph
4–6–8	91	70	51	81	59	48												
4–6–8	83	74	47	76	52	40	65	46	33	67	52	43						
4–6–12		72			61			59			62							
4–6–12		66			57			53			50							
4–9–8		73			60			53										
4–9–8		70			58			58										
4–9–12	83	68	47	79	61	47	77	55	46	74	59	50	56	35	33	63	46	36
4–9–12	81	67	50	77	61	46	79	49	40	66	51	43	54	32	29	52	41	32
5–6–8		62			60			54			59							
5–6–8		66			57			49			51			34			41	
5–6–12	92	70	46	79	60	46	84	59	41	71	62	51	57	39	39	61	47	38
5–6–12	85	68	46	75	60	40	80	50	38	73	53	48	47	42	30	53	41	34
5–9–8	91	67	47	83	67	45	79	60	46	69	66	53	57	42	34	62	48	37
5–9–8	83	57	48	79	56	48	76	57	48	76	54	54	53	38	25	57	42	37
5–9–12		73			61			66			70			40			50	
5–9–12		66			53			55			60			35			42	
8–3		59			52			55			54			40			44	
8–3		65			55			50			50			36			36	
8–6	77	53	40	79	53	43	74	55	40	73	55	43	65	37	35	68	45	38
8–6	79	58	42	71	55	42	68	48	35	70	52	35	56	35	28	59	39	31
9.5–3	81	58	44	75	52	46	77	55	43	75	57	44	64	38	35	75	52	40
9.5–3	81	61	48	70	56	43	71	53	36	74	54	43	59	34	28	65	38	31
9.5–6		53			50			54			57			40			50	
9.5–6		62			50			49			52			36			41	
8R–3	77	61	40	72	52	44	73	53	41	69	58	47	67	45	25	70	45	35
8R–3	83	55	40	73	57	45	67	46	35	67	50	38	56	39	27	59	37	29
8R–6		64			53			55			54			42			44	
8R–6		58			57			55			47			35				
9.5R–3		59			50			54			55			35			43	
9.5R–3		62			50			46			44			32			34	
9.5R–6	87	70	48	73	57	46	80	53	40	69	54	41	55	39	34	64	44	34
9.5R–6	91	73	47	73	59	37	73	55	38	69	39		52	33	25	53	35	29

[1] Surface thickness, base thickness, subbase thickness, all in inches.

TABLE 16–A

COEFFICIENT OF FRICTION FOR 30-KIP SINGLE AND 48-KIP TANDEM

Pavement Design[1]	Coefficient of Friction																	
	Series 1			Series 2			Series 3			Series 4			Series 5			Series 6		
	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph	10 Mph	30 Mph	50 Mph
5–6–12	68	73	57	77	52	40	62	43	43									
5–6–12	87	70	53	71	52	49	65	46	43	50	42	31						
5–6–16		68			56			43			40			34			44	
5–6–16		72			51			46			44			32				
5–9–12		68			56			39			40							
5–9–12		70			52			44			34							
5–9–16	66	70	57	77	56	39	71	50	38	64	47	34	53	35	30	60	41	35
5–9–16	85	66	50	72	53	43	69	52	35	56	47	32	47	40	29	53	37	31
6–6–12		68			55			49			44			34			41	
6–6–12		62			45			45			42			28			38	
6–6–16	68	70	53	72	57	38	72	54	38	76	54	41	49	35	28	71	51	31
6–6–16	87	69	50	77	55	46	69	52	39	62	47	30	48	32	24	53	39	29
6–9–12	70	68	55	79	56	40	66	49	38	62	46	41	53	36	27	56	43	34
6–9–12	89	71	51	72	55	49	69	49	40	56	46	30	49	35	27	59	41	33
6–9–16		74			56			38			45			32			40	
6–9–16		71			50			43			40			33			36	
9.5–3		66			55			52			43			35			39	
9.5–3		69			50			49			42			32			40	
9.5–6	77	55	53	70	49	40	68	51	39	62	45	32	54	38	30	68	40	33
9.5–6	77	57	37	62	45	33	64	47	34	57	45	27	49	37	26	57	38	30
11–3	84	64	59	70	53	43	72	50	39	62	44	34	52	38	30	60	44	32
11–3	74	64	57	65	51	36	64	41	40	52	38	29	52	34	26	54	38	29
11–6		58			52			52			48			37			45	
11–6		57			47			46			39			36			37	
9.5R–3	85	68	49	71	50	38	65	51	38	57	45	33	47	34	27	62	42	31
9.5R–3	73	70	59	63	45	38	56	43	36	52	33	27	45	32	26	52	37	30
9.5R–6		58			50			53			46			36			47	
9.5R–6		57			45			38			44			36			37	
11R–3		58			51			54			47			37			46	
11R–3		55			46			46			40			34			37	
11R–6	74	53	55	72	53	42	71	53	40	64	47	38	51	40	29	59	41	31
11R–6	70	59	55	67	55	36	67	48	38	56	45	29	51	38	26	55	36	30

[1] Surface thickness, base thickness, subbase thickness, all in inches.

TABLE 17-A

SUMMARY[1] OF COEFFICIENT OF FRICTION

Series	Speed (mph)	Coefficient of Friction																			
		Flexible		Rigid		Flexible		Rigid		Flexible		Rigid		Flexible		Rigid		Flexible		Rigid	
		2 Kips	6 Kips	2 Kips	6 Kips	12 Kips	24 Kips	12 Kips	24 Kips	18 Kips	32 Kips	18 Kips	32 Kips	22.4 Kips	40 Kips	22.4 Kips	40 Kips	30 Kips	48 Kips	30 Kips	48 Kips
1 Fall '58	10					0.90	0.90	0.83	0.82	0.83	0.85	0.88	0.91	0.89	0.83	0.81	0.84	0.68	0.87	0.80	0.74
	30	0.76	0.76	0.69	0.70	0.69	0.69	0.61	0.62	0.72	0.70	0.64	0.65	0.69	0.67	0.60	0.62	0.70	0.69	0.60	0.61
	50					0.54	0.52	0.49	0.45	0.55	0.52	0.48	0.50	0.48	0.48	0.43	0.44	0.56	0.51	0.54	0.52
2 Spr. '59	10	0.88	0.82	0.83	0.80	0.82	0.80	0.76	0.64	0.77	0.78	0.79	0.73	0.81	0.77	0.75	0.72	0.76	0.73	0.72	0.64
	30	0.63	0.56	0.58	0.50	0.59	0.53	0.52	0.40	0.63	0.55	0.51	0.46	0.61	0.57	0.52	0.55	0.56	0.52	0.52	0.44
	50	0.54	0.44	0.46	0.38	0.46	0.38	0.41	0.34			0.42	0.45	0.47	0.44	0.45	0.42	0.40	0.47	0.41	0.36
3 Sum. '59	10	0.84	0.77	0.83	0.73	0.76	0.69	0.72	0.61	0.75	0.71	0.77	0.67	0.80	0.75	0.76	0.70	0.68	0.68	0.89	.063
	30	0.62	0.56	0.57	0.54	0.55	0.46	0.51	0.41	0.58	0.51	0.55	0.47	0.58	0.51	0.54	0.50	0.46	0.47	0.52	0.45
	50	0.51	0.47	0.44	0.41	0.47	0.35	0.39	0.32	0.52	0.44	0.44	0.39	0.44	0.40	0.41	0.36	0.39	0.39	0.39	0.37
4 Spr. '60	10	0.86	0.75	0.81	0.79	0.73	0.63	0.73	0.70	0.84	0.67	0.74	0.62	0.71	0.71	0.72	0.70	0.67	0.56	0.61	0.54
	30	0.74	0.63	0.63	0.58	0.52	0.41	0.52	0.39	0.66	0.59	0.60	0.54	0.63	0.53	0.56	0.49	0.45	0.43	0.46	0.41
	50	0.59	0.56	0.52	0.48	0.41	0.35	0.36	0.30	0.49	0.34	0.44	0.35	0.51	0.47	0.44	0.37	0.39	0.31	0.34	0.28
5 Sum. '60	10	0.74	0.56	0.75	0.61	0.59		0.71	0.66	0.66	0.54	0.64	0.57	0.57	0.51	0.63	0.61	0.52	0.48	0.51	0.49
	30	0.59	0.47	0.52	0.43	0.47	0.41	0.46	0.41	0 44	0.35	0.44	0.39	0.39	0.36	0.40	0.35	0.34	0.33	0.37	0.35
	50	0.48	0.40	0.44	0.34	0.37		0.36	0.28	0.39	0.26	0.32	0.28	0.35	0.29	0.35	0.27	0.28	0.27	0.29	0.26
6 Fall '60	10	0.77	0.65	0.77	0.64	0.67		0.68	0.61	0.64	0.53	0.70	0.60	0.62	0.54	0.69	0.59	0.62	0.55	0.62	0.54
	30	0.65	0.52	0.59	0.44	0.48	0.43	0.47	0.39	0.51	0.41	0.51	0.43	0.48	0.42	0.46	0.37	0.44	0.38	0.43	0.38
	50	0.51	0.41	0.45	0.36	0.38		0.37	0.34	0.46	0.36	0.39	0.32	0.37	0.35	0.36	0.30	0.34	0.31	0.32	0.30

[1] Mean of pavement sections in Road Test at the time of the series.

Appendix B

REGIONAL ADVISORY COMMITTEES

These committees were appointed by the Highway Research Board to maintain liaison between the state highway departments and the research project, through the National Advisory Committee. Three members of each Regional Committee were appointed to the National Advisory Committee.

Region 1

F. M. Auer, Planning and Economics Engineer, New Hampshire Department of Public Works and Highways

E. B. Bly, Engineering Assistant to Commissioner, Vermont Department of Highways

T. V. Bohner, Special Assistant, Engineering Department, D. C. Department of Highways and Traffic

W. M. Creamer, Chief, Highway Staff Services, Connecticut State Highway Department

F. W. Hauck, Supervising Civil Engineer (Road Designing), Rhode Island Department of Public Works

C. D. Jensen, Director of Research and Testing, Pennsylvania Department of Highways

G. W. McAlpin, Assistant Deputy Chief Engineer (Research), New York State Department of Public Works

J. F. McGovern, Structures Maintenance Engineer, Massachusetts Department of Public Works

L. W. Novinger, Contract and Design Engineer, Delaware State Highway Department

V. A. Savage, Engineer of Primary Highways, Maine State Highway Commission

W. Van Breemen, Research Engineer, New Jersey State Highway Department

The following were members of the Region 1 Advisory Committee during the years indicated:

H. F. Clemmer, *formerly Chairman;* Consultant, D. C. Department of Highways and Traffic (1956-1960)

R. A. Farley, formerly Deputy Secretary, Engineering, Pennsylvania Department of Highways (1956-1958)

W. C. Hopkins, Deputy Chief Engineer, Maryland State Roads Commission (1956-1961)

F. S. Poorman, Deputy Secretary, Engineering, Pennsylvania Department of Highways (1959)

L. K. Murphy, formerly Construction Engineer, Primary Highways, Maine State Highway Commission (1955-1959)

Region 2

T. E. Shelburne, *Chairman,* Director of Research, Virginia Department of Highways

W. F. Abercrombie, Engineer of Materials and Tests, Georgia State Highway Department

T. L. Bransford, Engineer of Research and In-Service Training, Florida State Road Department

L. D. Hicks, Chief Soils Engineer, North Carolina State Highway and Public Works Commission

G. W. McAlpin, Director, Program Office, and Assistant Chief Engineer, West Virginia State Road Commission

J. D. McMahan, Construction Engineer, South Carolina State Highway Department

A. O. Neiser, Assistant State Highway Engineer, Kentucky Department of Highways

T. W. Parish, Assistant Chief Engineer (Construction), Louisiana Department of Highways

R. S. Patton, Engineer of Surveys and Designs, Tennessee Department of Highways and Public Works

Angel (2) Silva, Director, Puerto Rico Department of Public Works

H. O. Thompson, Testing Engineer, Mississippi State Highway Department

J. F. Tribble, Materials and Research Engineer, Alabama State Highway Department

E. L. Wales, Engineer of Materials and Tests, Arkansas State Highway Commission

The following was a member of the Region 2 Advisory Committee during the years indicated:

J. L. Land, formerly Chief Engineer, Bureau of Materials and Tests, Alabama State Highway Department (1956)

Region 3

W. E. Chastain, Sr., *Chairman,* Engineer of Physical Research, Illinois Division of Highways

J. G. Butter, Consultant, Iowa State Highway Commission

E. A. Finney, Director, Research Laboratory, Michigan State Highway Department

R. A. Helmer, Research Engineer, Oklahoma State Highway Department

J. W. Hossack, State Engineer, Nebraska Department of Roads

C. P. Jorgensen, Manager, Research and Planning, South Dakota State Highway Commission

H. E. Marshall, Research Engineer, Ohio Department of Highways

R. L. Peyton, Assistant State Highway Engineer, State Highway Commission of Kansas

J. S. Piltz, Engineer of Design, Wisconsin State Highway Commission

C. K. Preus, Materials and Research Engineer, Minnesota Department of Highways

F. V. Reagel, Engineer of Special Assignments, Missouri State Highway Commission

W. T. Spencer, Soils Engineer, Indiana State Highway Department

W. A. Wise, Director, Field Division, North Dakota State Highway Department

The following were members of the Region 3 Advisory Committee during the years indicated:

L. N. Ress, formerly State Engineer, Nebraska Department of Roads (1956-1958)

H. G. Schlitt, formerly Deputy State Engineer, Nebraska Department of Roads (1959)

C. W. Allen, formerly Research Engineer, Ohio Department of Highways (1956-1958)

J. H. Swanberg, Chief Engineer, Minnesota Department of Highways (1956-1958)

Region 4

R. E. Livingston, *Chairman,* Planning and Research Engineer, Colorado Department of Highways

J. R. Bromley, Superintendent and Chief Engineer, Wyoming State Highway Department

L. F. Erickson, Assistant Construction Engineer, Idaho Department of Highways

L. B. Fox, Construction Engineer, Montana State Highway Commission

T. S. Huff, Chief Engineer of Highway Design, Texas State Highway Department

F. N. Hveem, Materials and Research Engineer, California Division of Highways

C. W. Johnson, Materials and Testing Engineer, New Mexico State Highway Commission

D. F. Larsen, Chief Materials Engineer, Utah State Road Commission

C. E. Minor, Materials and Research Engineer, Washington Department of Highways

W. G. O'Harra, Materials Engineer, Arizona Highway Department

W. M. Wachter, Highway Engineer, Hawaii Division of Highways

W. O. Wright, State Highway Engineer, Nevada Department of Highways

The following were members of the Region 4 Advisory Committee during the years indicated:

W. T. Holcomb, formerly Assistant State Highway Engineer, Nevada Department of Highways (1956-1959)

I. B. Miller, Operations Engineer, New Mexico State Highway Commission (1956-1958)

B. E. Nutter, formerly Territorial Highway Engineer, Hawaii Territorial Highway Department (1956-1958)

S. B. Sanders, formerly District Engineer, Montana State Highway Commission (1956-1958)

W. C. Williams, State Highway Engineer, Oregon State Highway Commission (1956-1961) (deceased)

ADVISORY PANEL ON SPECIAL STUDIES

This panel assisted in formulating a series of studies related to the test vehicles, vehicle drivers, and heavy military vehicles, to be conducted during and after the traffic phase of the AASHO Road Test, and to include both non-destructive and destructive testing of the pavements and bridges.

E. H. Holmes, *Chairman,* Assistant Commissioner for Research, Bureau of Public Roads

R. R. Bartelsmeyer, Chief Highway Engineer, Illinois Division of Highways

D. S. Berry, Professor, Northwestern University

P. P. Brown, Assistant Consultant for Soil Mechanics and Paving, Bureau of Yards and Docks, Department of the Navy

E. L. Erickson, Chief, Bridge Division, Bureau of Public Roads

B. H. Fox, Assistant Chief, Accident Prevention Program, Public Health Service, Department of Health, Education and Welfare

F. B. Hennion, Assistant Chief, Airfield Branch, Office of the Chief of Engineers, Department of the Army

Col. C. W. Hill, Chief, Neuropsychiatry and Psychophysiology Research Branch, U. S. Army Medical Research and Development Command

Maj. R. A. Hoffman, Chief, Transportation Vehicle Section, Artillery and Vehicle Systems Branch, Research and Development Division, Office of the Chief of Ordnance, Department of the Army

F. N. Hveem, Materials and Research Engineer, California Division of Highways

R. C. Kerr, Consultant to the Chief of Transportation, Office of the Chief of Transportation, Department of the Army

C. H. Perry, Deputy Director of Transportation Engineering, Office of the Chief of Transportation, Department of the Army

K. M. Richards, Manager, Field Services Department, Automobile Manufacturers Association

J. E. Uhlaner, Director of Research Laboratories, U. S. Army Personnel Research Office, Office of the Chief of Research and Development

The following was a member of this panel during the years indicated:

C. A. Weber, Chief Engineer, Michigan State Highway Department (Resigned 1960)

SPECIAL PUBLICATION SUBCOMMITTEE FOR AASHO ROAD TEST REPORT 6, SPECIAL STUDIES

This subcommittee was appointed by the Highway Research Board to advise the project staff in the preparation of AASHO Road Test Report 6, "Special Studies", and recommend approval of the report for publication.

E. H. Holmes, *Chairman,* Assistant Commissioner for Research Bureau of Public Roads

D. K. Chacey, Director of Transportation Engineering, Office of the Chief of Transportation, Department of the Army

W. E. Chastain, Sr., Engineer of Physical Research, Illinois Division of Highways

R. E. Fadum, Head, Civil Engineering Department, North Carolina State College

C. E. Fritts, Vice-President for Engineering, Automotive Safety Foundation

Maj. R. A. Hoffman, Chief, Transport Vehicle Section, Office of the Chief of Ordnance, Department of the Army

J. B. Hulse, Managing Director, Truck Trailers Manufacturers Association

W. C. Johnson, Manager, Tire Test Division, Goodyear Tire and Rubber Company; acting for the Tire and Rim Association

L. C. Lundstrom, Director, General Motors Proving Ground; Automobile Manufacturers Association

R. A. Moyer, Professor, Institute of Transportation and Traffic Engineering, University of California

T. E. Shelburne, Director, Highway Investigation and Research, Virginia Department of Highways

H. O. Thompson, Testing Engineer, Mississippi State Highway Department

PROJECT PERSONNEL

Project Staff and Engineers

W. B. McKendrick, Jr., Project Director
W. N. Carey, Jr., Chief Engineer for Research
Peter Talovich, Business Administrator
L. A. Ptak, Accountant
R. S. Semple, Purchasing Assistant
A. C. Tosetti, Assistant to the Project Director
W. R. Milligan, Assistant Operations Manager
D. L. Thorp, Shop Superintendent
A. C. Benkelman, Flexible Pavement Research Engineer
H. M. Schmitt, Assistant Flexible Pavement Research Engineer
F. H. Scrivner, Rigid Pavement Research Engineer
W. R. Hudson, Assistant Rigid Pavement Research Engineer
R. J. Little,[1] Assistant Rigid Pavement Research Engineer
I. M. Viest, Bridge Research Engineer
J. W. Fisher, Assistant Bridge Research Engineer
P. E. Irick, Chief, Data Processing and Analysis
R. C. Hain, Assistant Chief, Data Processing and Analysis
J. F. Shook, Materials Engineer
D. R. Schwartz,[1] Engineer of Reports
H. R. Hubbell,[1] Assistant Engineer of Reports
H. H. Boswell, Maintenance Engineer
R. C. Leathers, Engineer of Special Assignments
A. J. Wright,[1] Assistant Engineer of Special Assignments
H. Y. Fang,[1] Assistant Engineer of Special Assignments
H. C. Huckins, Supervisor, Instrument Laboratory
W. J. Schmidt, Chief, Public Information

[1] Illinois Division of Highways

U. S. Army Transportation Corps Road Test Support Activity (AASHO)

Commanding Officer
Col. A. A. Wilson (1958-59)
Lt. Col. R. J. Lombard (1959-61)

Deputy Commander
Maj. W. A. Duncan (1958-60)
Capt. R. G. Farwell (1960-61)

Company Commander
Capt. R. D. Smith (1958-59)

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

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The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY-COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.