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Flexible Pavement Design And Performance Studies

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Flexible Pavement Performance Studies in Arkansas

MILLER C. FORD, Jr. and J. R. BISSETT, Respectively, Assistant Professor of Civil Engineering and Professor of Civil Engineering, University of Arkansas

This paper reports the test methods used and results obtained in evaluating the performance of 115 mi of flexible pavement. The pavement varies in age from three to eight years, from high-type asphaltic concrete in excellent condition to double surface treatments that have required extensive maintenance. All of the roads are on the same soil area and have similar climatic conditions. The deflections were measured by the Benkelman beam with Helmer recorder. Seven series of deflections were made over a period of two years at some 500 different stations. In-place densities and moisture contents were determined at about one-third of these stations. Also moisture-density samples of the subgrade were taken at the edge of the pavement at different seasons to study the variations of moisture and density with time. Physical properties of subgrade and base materials are also reported. Visual condition surveys were made at frequent intervals during the life of the project to detect changes in the pavement condition.

● THIS REPORT is part of a study of the performance of flexible pavement being conducted by the University of Arkansas in cooperation with the Arkansas Highway Department and the Department of Commerce, Bureau of Public Roads. The study was started in July 1958 on roads in the loess-terrace soil area located in eastern Arkansas. The purpose of the study is to evaluate 115 mi of pavement, relating performance with pavement deflection; physical properties of subgrade, base and pavement; engineering and agricultural soil classifications; maintenance required; and amount of traffic. The final goal is the development of a better method of design of flexible pavements. This paper reports the tests performed in evaluating pavement structure and the results of these tests. The information obtained on the asphalt pavement is reported in J. R. Bissett's paper, "Changes in Physical Properties of Asphalt Pavement with Time," HRB Proceedings, Vol. 41.

Physical properties of the pavement structure were determined by taking samples of the pavement, base, and subgrade at about every 3/4 mi along the study roads.

The pavement sample was secured from the centerline of the traffic lane. The base density was measured below this pavement sample and a sample of the subgrade was taken at this point and at the edge of the pavement. The subgrade samples were taken with a thin-wall sampling tube 3 in. in diameter by 10 in. long. The thickness of each layer encountered during sampling plus depths to subgrade samples were recorded.

The pavement deflections were measured with a Benkelman beam (Fig. 1); a Helmer recorder (Fig. 2) provided a graph of each deflection. In addition the maximum and final deflections were taken from the dial gage.

Deflections were obtained from seven series of tests along the study roads. There are about 500 locations where the tests were run, and both the inner and outer wheel deflections were measured each time, making a total of about 7,000 deflections which were used in preparing data for this paper.

ROADS UNDER STUDY

The roads under study range from high-type asphaltic concrete pavements in excellent

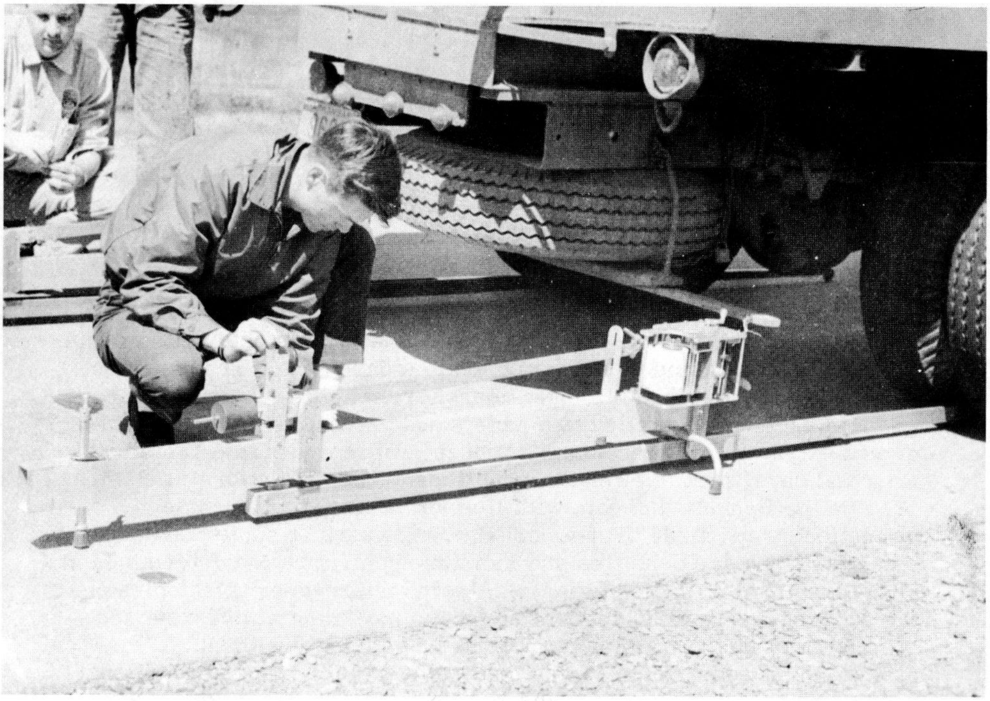

Figure 1.

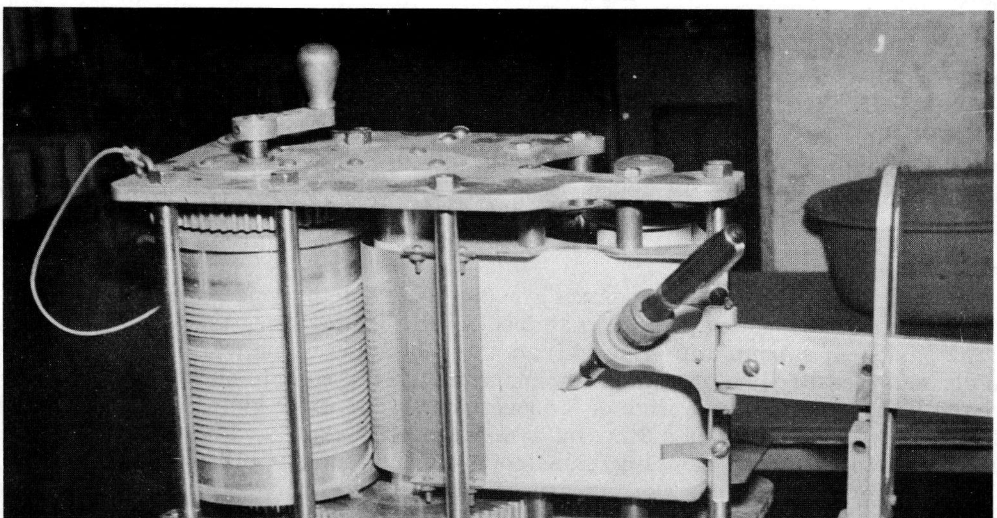

Figure 2.

condition to double surface treatments that have required almost complete rebuilding. Their age is from three to eight years, and they were built to Arkansas Highway Department specifications. They were chosen to represent the various types of pavement that have been constructed in Arkansas during the past few years. Each individual road will be referred to hereafter as a "job."

Seven of the jobs are hot-mix asphaltic concrete pavement on gravel bases. Jobs I, J, and M are pavements made with crushed aggregate, and Jobs A, B, C, and F are pavements made with local gravel that was crushed to fit gradation requirements. The total length of these seven jobs is 60 mi. The hot-mix asphaltic concrete pavements are grouped as high-type pavements.

The remaining seven jobs totaling 55 mi in length will be grouped as low-type pavements in these discussions. Jobs D, G, H, K, L, and N are double surface treatments and all are laid on gravel bases except Job K which is laid on a crushed rock base. Job E is a road mix, laid on a gravel base.

No further mention will be made of Jobs C, D, and G for which data are not complete at this time.

SAMPLING

Station numbers were painted along each road at about every 0.2 mi to be used as reference points in carrying out the study. The pavement sample consisted of a 15- by 15-in. square cut from the centerline of the traffic lane, using an air hammer. The pavement was removed without disturbing the base. The density of base was measured with a balloon density apparatus. The base sample was placed in a syrup bucket and returned to the laboratory for drying and weighing. About 1/2 gal of the base material was secured for determination of the index properties. A thin-wall sampling tube was driven into the subgrade to obtain a sample. The ends of the tube were sealed with paraffin. The depths to the tube sample and thickness of subbase, base, and pavement were recorded at the time the sample was taken. A hole was dug at the edge of the pavement through the base and into the subgrade. A gallon bucket of this subgrade soil was taken for running the maximum density test. Finally a tube sample of the subgrade was taken at the edge of pavement for determination of in-place density and moisture.

PHYSICAL TESTS OF BASE AND SUBGRADE

The material removed from the holes in the base was placed in a 1-gal syrup bucket and transferred to the laboratory where the unit dry weight and moisture were determined. To avoid loss of moisture or parts of the sample, the weight of the bucket and material was obtained before the bucket was opened. The bucket was then opened and the material dried in an oven. A new type of ballon density apparatus having a vacuum-pressure pump with a pressure gage was used in determining the volume of the hole.

The volume of the soil in the thin-wall sampling tube was obtained by measuring the length of the sample before it was removed from the tube. The weight of the wet soil was determined, then the sample was removed from the tube and a representative sample taken to determine the moisture content. The unit dry weight was then determined.

The dry method of preparation of samples was used to prepare base and subgrade samples for testing.

The averages of the test results are given in Table 1 for both high- and low-type pavements. All the index properties of both the base and the subgrade were determined using the appropriate AASHO standard method. The maximum density and optimum moisture contents were determined by Method AASHO T 180-57 (Method A).

Present base and subgrade densities show considerable uniformity. There is no reason to believe that these densities were not higher at the time they were constructed. The only conclusion is that an increase in moisture of the subgrade has caused the densities to decrease. The majority of these jobs are on a flat plain where surface drainage is very poor; in fact in most cases, the roadside ditches are full of water throughout the year. Most of the loess-terrace soil area is underlain with a clay pan that is about 50 in. below the surface, and only Jobs A and F have good surface drainage.

The plasticity index of the high-type pavement subgrade varies from 5 to 8 except

TABLE 1

AVERAGE PHYSICAL PROPERTIES

Pavement	Thickness (in)		Density				Moisture (%)		Liquid Limit, Subgrade	Plasticity Index, Subgrade	Silt Size, Subgrade (%)	Clay Size, Subgrade (%)
	Pave-ment	Base	Base		Subgrade		Subgrade	Subgrade Opt.				
			Max. (pcf)	In-Place (%)	Max. (pcf)	In-Place (%)						
High-type												
A	2.2	6.5	131	96	120	88	17	14	29	8	68	23
B	1.9	7.4	130	94	119	84	20	14	37	15	44	39
F	1.8	6.9	133	93	120	88	15	15	30	8	61	28
I	2.0	7.0	138	89	117	87	17	13	32	7	67	26
J	1.9	9.5	132	92	120	83	16	13	28	6	56	22
M	2.2	9.9	138	91	126	81	17	11	30	5	56	20
Low-type												
E	2.2	5.2	136	93	118	86	18	14	26	3	64	25
H	-	3.8	137	89	110	87	21	13	32	12	66	26
K	0.7	8.5	137	91	121	84	17	13	31	6	62	27
L	0.9	6.0	133	94	120	85	17	11	33	12	62	29
N	0.6	6.1	136	93	120	87	20	14	34	12	62	27

for Job B which has a plasticity index of 15. The subgrade plasticity index for the low-type pavements vary from 3 to 12, with Jobs H, L, and N having a plasticity index of 12.

The base index properties are not tabulated because they were very nearly the same for all jobs. This material is a clay gravel, well graded, having a plasticity index from 0 to 3. This material can be compacted into a rather dense material as indicated from the maximum base densities given in Table 1.

The base thickness (Table 1) was taken from the measurement made at the center of the traffic lane. The average for high-type pavements was 7.9 in., with Job M being thickest with 9.9 in. The base under the low-type pavement averaged 5.9 in., with Job K being thickest with 8.5 in. and Job H being thinnest with 3.8 in. However, Job H had a sandy subbase material that varied in thickness from 3 to 7 in.

DEFLECTION TESTS

Two Benkelman beams were used simultaneously for making the pavement deflection tests. These two beams were equipped with the Helmer recorder, so that curves of the deflections as well as the gage readings were determined at point of maximum deflection. In all cases, the loading truck wheel was 4 ft to the rear of the probe point at the beginning of the test. The truck was then moved forward at the slowest possible rate until the wheel of the truck was at least 6 ft beyond the probe.

The loading truck was equipped with two water tanks so constructed that the water could be shifted from one tank to the other by means of a pump. The actual wheel loads were maintained at 9,000 lbs each and the tire pressure was maintained at 90 psi. Loadometers were used to determine the weight of the truck wheels.

Figure 3 shows curves typical of the average and maximum deflection obtained on Job I. In these curves, the Helmer graph shows a horizontal line for some distance before the deflection started. This is in agreement with field tests in which the truck was placed some 30 or 40 ft forward of the beam probes and then backed to the probes. In all cases the dial gages on the beams did not indicate any deflection until the wheel was within from 2 to 3 ft of the probe.

There is some question whether the recovery part of the Benkelman beam curve shown by the Helmer recorder is accurate because of frictional resistance between the recording pen and the paper. There is also some flexibility in the recording beam. The recording beam was constructed so that the pressure of the pen on the paper was adjustable. This pressure was reduced to the minimum that would mark the paper, yet in test it was found that the gage on the beam did not return to zero after the truck had been moved beyond the zone of influence. When the recording pen was removed from the device and the recording beam permitted to swing freely, the dial gage

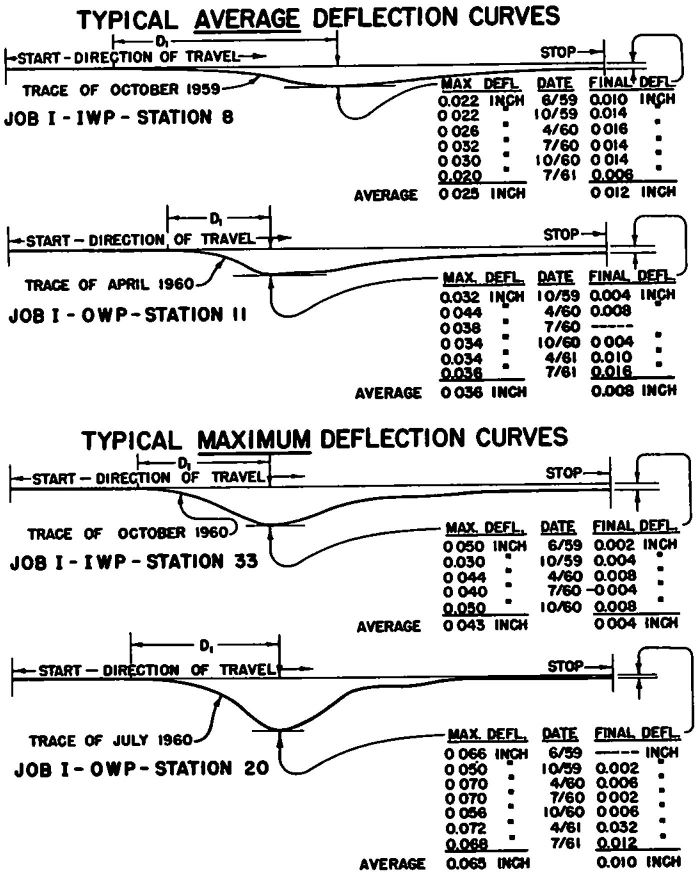

Figure 3. Typical deflection curves, Benkelman beam with Helmer recorder, 9,000-lb wheel load. Scale: Horiz. 1 in. = 1 ft - vert. 1 in. = 0.10 in.

invariably returned to zero or within 0.001 of the initial starting place.

Table 2 gives average deflections for all of the jobs and for the seven series of deflections. In most cases the deflection of the high-type pavement was low and rarely did the average of the deflection of the inner wheel exceed 0.03 in. or the outer wheel exceed 0.04 in. in every series of deflection tests. There were a few erratics due to pavement conditions where the test was made. The deflection was always made at the same location at each station by using a line painted on the pavement and the truck wheel was stopped on this line for the beginning of the deflection measurement. The lateral location of the truck wheel did not vary more than about 6 in. for any station, and ranged from 18 to 24 in. from the edge of the pavement.

A wide variation in the deflection at the same station was noted at different times. Figure 3 shows these variations. For example, for Job I on the outer wheelpath at station 20, the average of the maximum deflections is 0.065 in., but the range is from 0.050 to 0.072 in. No reason has been discovered for these erratics. Usually the deflections followed a fairly uniform pattern.

In most cases there was a residual deflection as explained previously. To illustrate, on Job I, for the outer wheelpath, at station 20 the average of the residual deflection was 0.010 in., the minimum residual was 0.002 in. and maximum residual 0.032 in. Checks and investigations have convinced that most of this residual deflection is due to the friction of the pen on the paper and, possibly, some to the flexibility of the recording arm of the beam. Frequently the truck was moved some distance beyond the end of the probe at the end of the test and allowed to stand for several minutes. The dial gage on the beam did not indicate any change in the deflection, even after several minutes. It is felt that if residual deflection existed in the pavement under each truck load, a severe rutting would be evident but this is not the case.

HIGH-TYPE PAVEMENT DEFLECTION

The average deflection obtained for each job from the seven series of tests is shown in Figure 4. The jobs are placed left to right in order of their pavement age. Computed to July 1960 (Table 3). Other data given in Table 3 are daily traffic, equivalent wheel loads per day, total equivalent wheel loads, average condition surveys, selected deflection data, radius of influence, and ratio of radius of influence to deflection.

The outer wheel deflection on the average exceeded the inner wheel deflection by about 40 percent. This differential deflection is believed to be caused primarily by the lack of confining support from the shoulder. Job M has the widest shoulders and has a differential deflection of only 0.004 in., and Job B has the narrowest shoulders and has a differential deflection of 0.015 in.

Figure 4 indicates that the youngest job (Job M) had the lowest deflection, the middle-aged pavements had a higher deflection (Jobs F, J, and I) and then the oldest pavements (Jobs A and B) decreased in deflection below that of the middle pavements and were slightly higher than the younger pavement.

TABLE 2

AVERAGE DEFLECTIONS

Date	Deflection (10^{-3}in.)													
	Job A		Job B		Job F		Job I		Job J		Job M		Average	
	IWP	OWP	IWP	OWP	IWP	OWP	IWP	OWP	IWP	OWP	IWP	OWP	IWP	OWP
(a) High-Type Pavement														
June '59	11	21	17	33	17	23	28	44	24	35	21	28	20	31
Oct. '59	19	31	26	37	24	30	30	42	23	29	19	25	23	33
April '60	13	33	18	46	18	35	34	48	24	34	22	28	21	37
July '60	19	32	20	36	18	23	30	43	18	28	18	21	20	30
Nov. '60	18	18	26	27	27	23	32	34	26	21	18	17	24	23
April '61	15	30	20	41	19	36	29	37	19	24	14	21	19	31
July '61	15	24	22	34	17	20	33	40	28	27	17	15	22	27
Average	16	27	21	36	20	27	31	41	23	28	18	22	21	30

(b) Low-Type Pavement

	Job E		Job H		Job K		Job L		Job N		Average	
	IWP	OWP	IWP	OWP	IWP	OWP	IWP	OWP	IWP	OWP	IWP	OWP
June '59	36	27	31	41	20	38	17	28	20	32	25	33
Oct. '59	43	43	38	45	20	32	26	36	29	40	31	39
April '60	49	41	33	37	19	34	19	42	25	49	29	41
July '60	39	32	29	40	18	23	18	23	11	26	23	29
Nov. '60	41	33	32	35	21	19	23	24	24	26	28	27
April '61	44	39	37	47	21	28	16	30	18	41	27	37
July '61	35	23	37	42	21	20	17	27	21	27	26	28
Average	41	34	34	41	20	28	19	30	21	34	27	33

Figure 5 shows the variation of deflection of both inner and outer wheel for each series of test run on Job A. Comparison with Figure 6 plotted for Job I shows quite a large difference between these two jobs in over-all deflection, but attention is called to the uniform deflection of the inner wheel with season, while the outer wheel fluctuates seasonally. It has been observed that the highest deflections occur during the spring tests for the outer wheel and during the fall tests for the inner wheel. However, there is not a great amount of difference in the average deflections; with the inner wheel ranging from 0.019 to 0.024 in. and the outer wheel ranging from 0.027 to 0.037 in.

Figures 7 and 8 show the variation of deflections within a single job for the outer wheel. Figure 7 is for Job I, the maximum deflection occurs at station 27, where the pavement has a large longitudinal crack, and the deflection is 0.094 in. The second highest deflection occurs at station 36 and amounts to 0.084 in. There is no apparent reason for such a high deflection at this location. The minimum deflection for this job occurs both at station 14 and 45 in the amount of 0.015 in. Deflections along Job A are shown on Figure 8. The range in deflection is less than for Job I; the maximum

TABLE 3

MISCELLANEOUS DATA

Pavement	ADT 1960 (VPD)	No of Equiv Wheel Loads[1]		Condition Survey (%)			Pavement Age July 1960 (yr)	Average Deflection (in.)		Avg Radius of Influence (ft)		Radius/ Deflection (in /in)	
		Per Day	Total (thousands)	Average	Highest	Lowest		IWP	OWP	IWP	OWP	IWP	OWP
A	1,000	401	650	88	95	86	8 1	0 017	0 025	2 3	2 4	1,623	1,152
B	1,700	796	1,743	66	70	62	8.1	0 020	0 039	2 4	2 5	1,440	769
F	1,300	401	694	70	81	60	5 7	0 020	0.030	1 9	1 9	1,140	633
I	2,100	1,531	3,129	88	95	81	6 7	0 031	0.043	2 5	2 5	968	465
J	2,150	1,856	3,048	97	98	95	5.2	0 027	0.032	2 6	2 4	1,156	900
M	2,100	1,856	1,328	98	98	97	2 1	0 021	0 025	2 4	2 2	1,371	1,056
E	900	209	459	69	83	60	7 0	-	-	-	-	-	-
H	450	126	103	80	89	66	2 9	-	-	-	-	-	-
K	575	195	289	77	80	75	4 9	-	-	-	-	-	-
L	550	126	188	82	95	68	5.6	-	-	-	-	-	-
N	325	126	106	82	85	68	2 7	-	-	-	-	-	-

[1]5,000 lb.

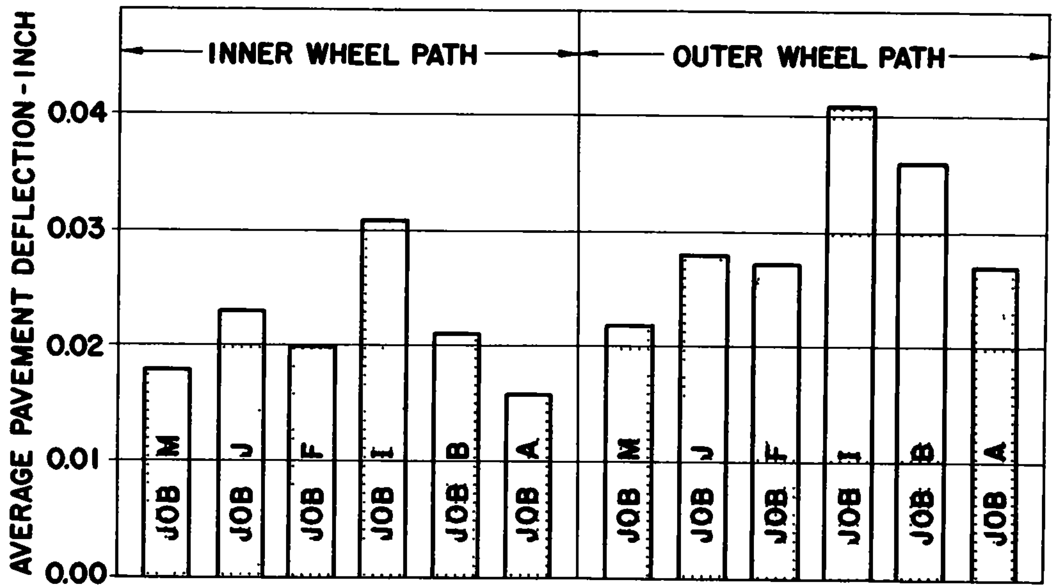

Figure 4. Average pavement deflection vs job, Benkelman beam, 9,000-lb wheel load, high-type pavement.

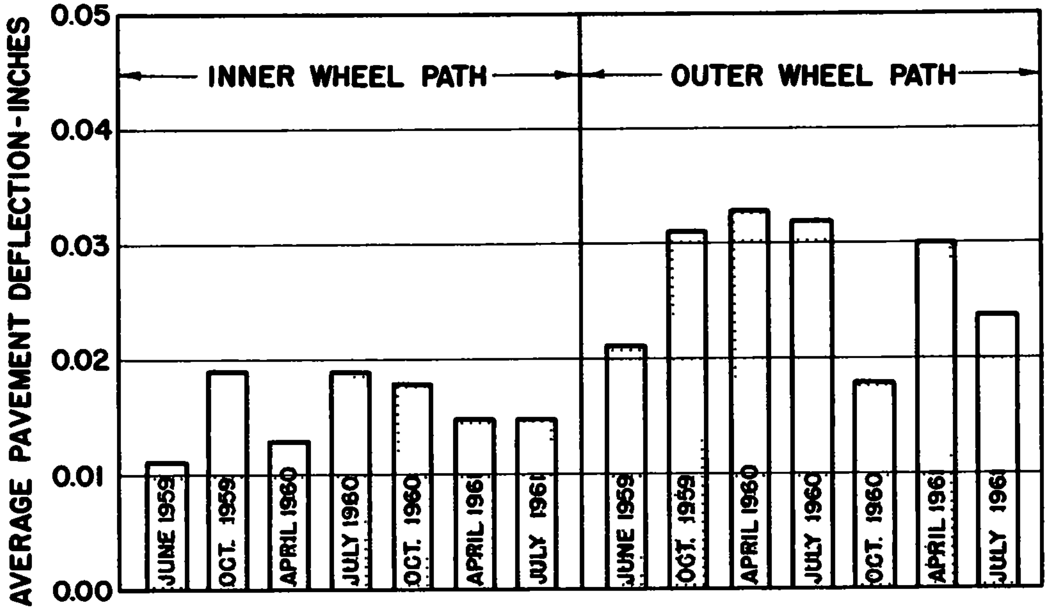

Figure 5. Average deflection vs test date, Job A, Benkelman beam, 9,000-lb wheel load.

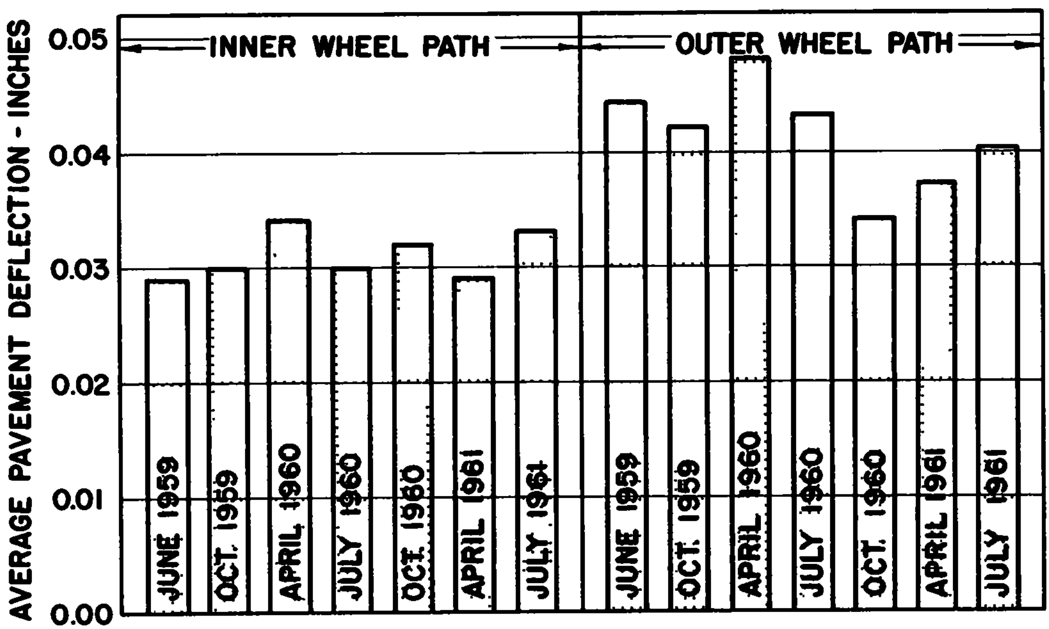

Figure 6. Average deflection vs test date, Job I, Benkelman beam, 9,000-lb wheel load.

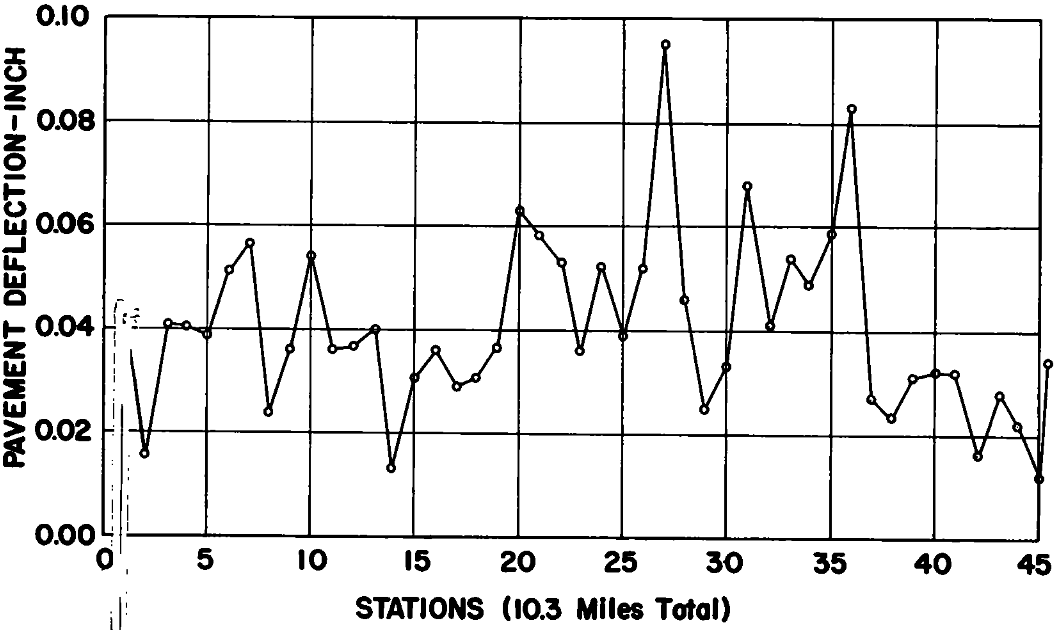

Figure 7. Average deflection vs location, Job I, Benkelman beam, outer wheelpath.

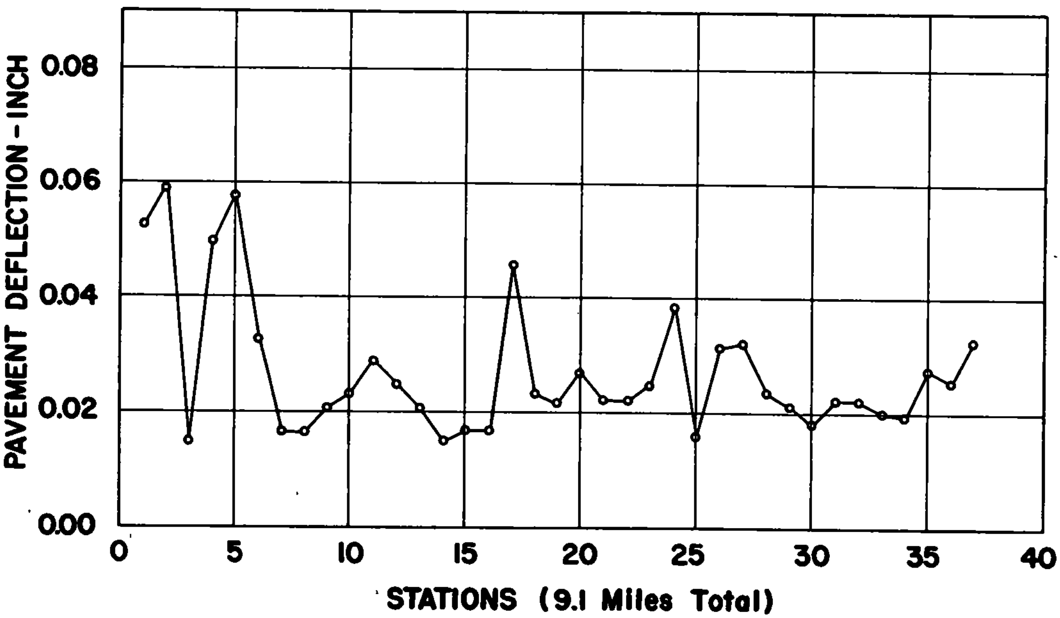

Figure 8. Average deflection vs location, Job A, Benkelman beam, outer wheelpath.

deflection occurs at station 2 and totals 0.059 in., with the minimum deflection occurring at station 3 in the amount of 0.015 in.

Figure 9 shows the pavement deflections plotted against the total thickness of pavement and base for both the inner and outer wheels of Job M. There is a definite decrease in pavement deflection with thickness of structure for most jobs. This trend is not very well indicated where the average deflections were comparatively low, however. For Job M the inner wheel has the more positive trend, indicating a deflection of 0.045 in. with a structure thickness of 8 in., varying to a deflection of 0.008 in. with a structure thickness of 16 in.

Results of the analysis of the deflection curves obtained from the Helmer recorder are shown in Table 3. The radius of influence of the wheel is assumed to be from the point of maximum deflection back to where the curve becomes tangent to the horizontal. This radius of influence as defined is shown as d_1 in Figure 3. Measurement of this radius of influence shown on the graph given by the Helmer recorder has been completed on selected stations of the high-type pavement only. The radius of influence varies from 1.9 ft on Job F to 2.6 ft on Job J. It is noted that the deflections given in Table 3 are average deflections for selected stations and are not to be confused with the average deflections given in Table 2.

The ratio of radius of influence to pavement deflection is calculated in units of inches of radius of deflection to inches of pavement deflection (in./in.). The average of both inner and outer wheel ratios range from 1,387 on Job A to 716 on Job I. It is noted that the higher the ratio of influence to deflection for a single job was the larger the zone of influence indicated or the smaller the deflection occured. It is felt that the ratio for outer wheelpath is more indicative of the pavement structure condition, and with a ratio less than 800 the pavement is in poor condition.

Again grouping the pavements into three groups based on their age and averaging their outer wheel ratio of influence to deflection gives an interesting comparison. The youngest Job M's ratio is 1,152, the middle-aged pavements Jobs F, I, and J's ratio is 666, and the older pavement for Job B's ratio is 960.

Table 4 gives the criteria followed in evaluating the condition of the pavement. The average condition shown is a percent based on a new pavement having 100 percent

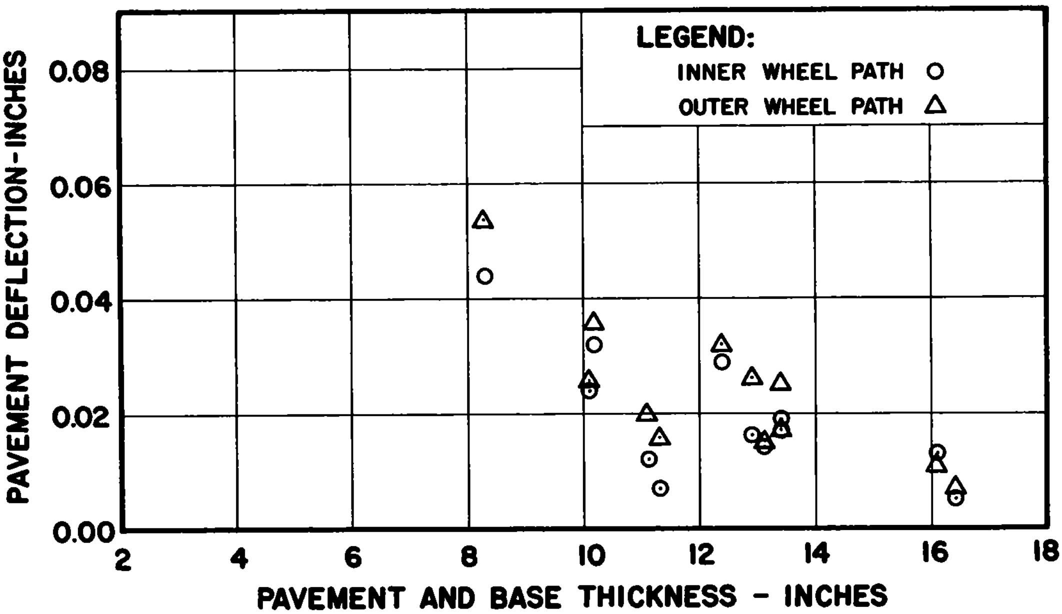

Figure 9. Deflection vs pavement structure, Job M.

TABLE 4

CRITERIA FOR CONDITION SURVEYS

EXCELLENT 95-100
- No defects apparent
- Good riding surface

GOOD 90-95
- Few small isolated cracks
- Slight surface roughness
- No patching required

FAIR 80-90
- Some isolated cracks
- Slight surface irregularities
- Some raveling at edge of pavement

AVERAGE 70-80
- Slight rutting
- Small areas showing map cracking
- Small raveled areas
- Minor base failures
- Surface roughness evident

POOR 55-70
- Distorted surface
- Base failures extend entire width of lane
- Considerable surface cracking
- Rutting

FAILURE below 55
- Extensive patching
- Surface distortion
- Extensive base failures

condition. Seven condition surveys have been completed. Each segment of road between stations is evaluated from visual observations and the average per job determined. The maximum, average, and minimum percent condition surveys are given in Table 3.

The percent condition varied from survey to survey, with the maintenance work performed increasing the pavement rating. The lowest condition rating may be the best comparison between jobs in over-all performance. The lowest ratings vary from 60 percent on Job F to 97 percent on Job M.

The traffic data given in Table 3 was prepared by the planning and research staff of the Arkansas Highway Department. All wheel loads are converted into equivalent 5, 000- lb wheel loads. No definite relationship has been established between loading and percent condition or pavement deflection. The average daily traffic varies from 1, 000 to 2, 100 vehicles per day.

LOW-TYPE PAVEMENT DEFLECTION

The bar graph in Figure 10 compares the inner and outer wheel deflections obtained.

Only Job E showed a higher deflection in the inner wheel path than in the outer wheel path. This particular job is in very bad condition. No explanation has been found for this unusual behavior. In fact most of this job has required resurfacing or rebuilding during the period of these tests. Table 2 gives the total number of equivalent 5, 000- lb loads for this job as 459, 000, considerably more than for any other low-type pavement. The average deflections on this pavement were one of the two highest studied.

Job H also shows very high deflections. This job has failed almost completely and been rebuilt. The thickness of the base varied widely from station to station. However, there was about 5 in. of sandy subbase under the base material.

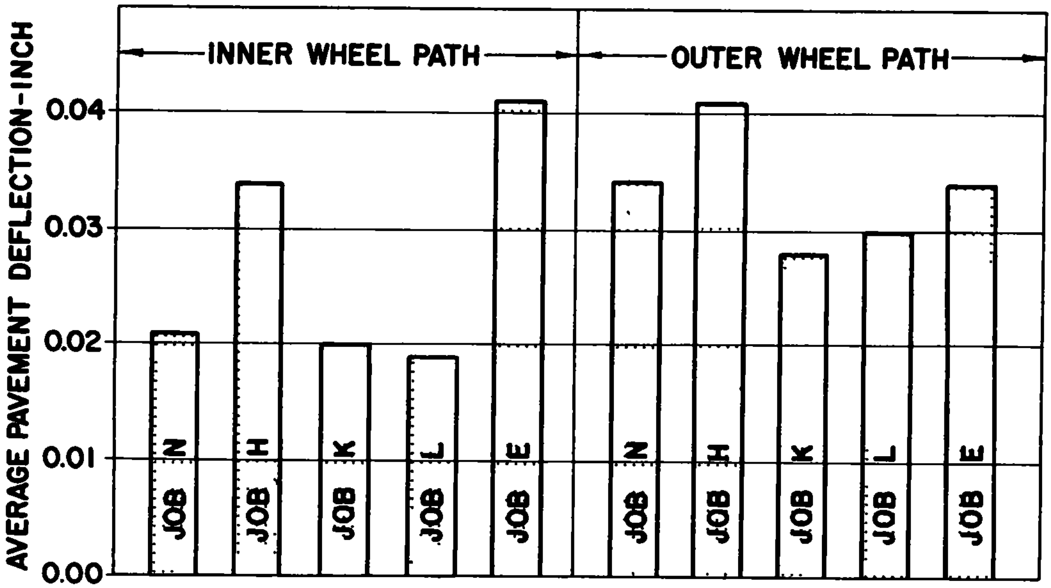

Figure 10. Average pavement deflection vs job, Benkelman beam, 9,000-lb wheel load, low-type pavement.

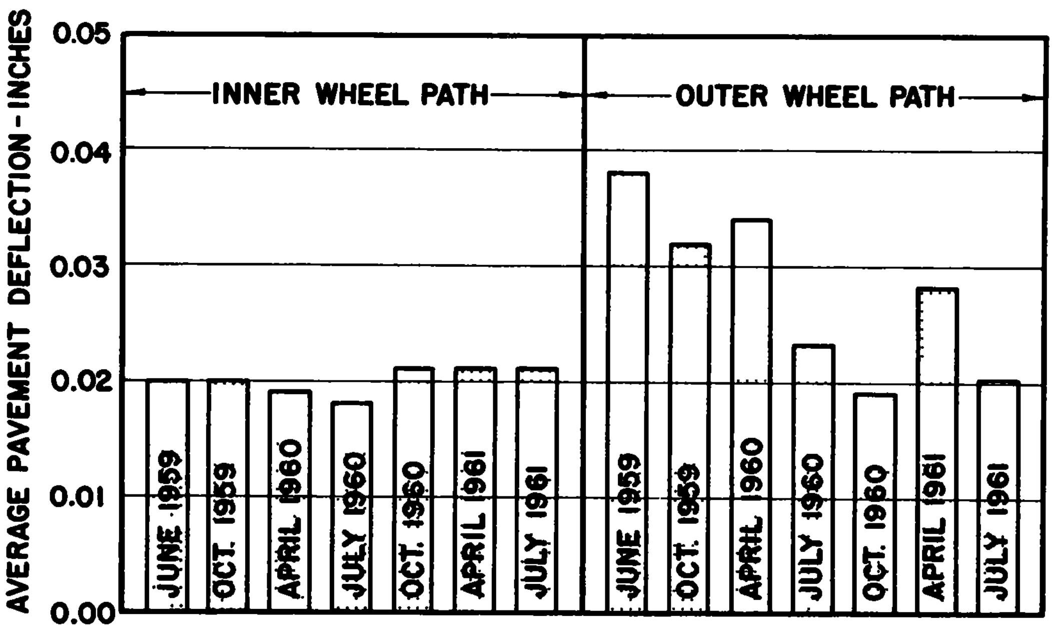

Figure 11. Average deflection vs test date, Job K, Benkelman beam, 9,000-lb wheel load.

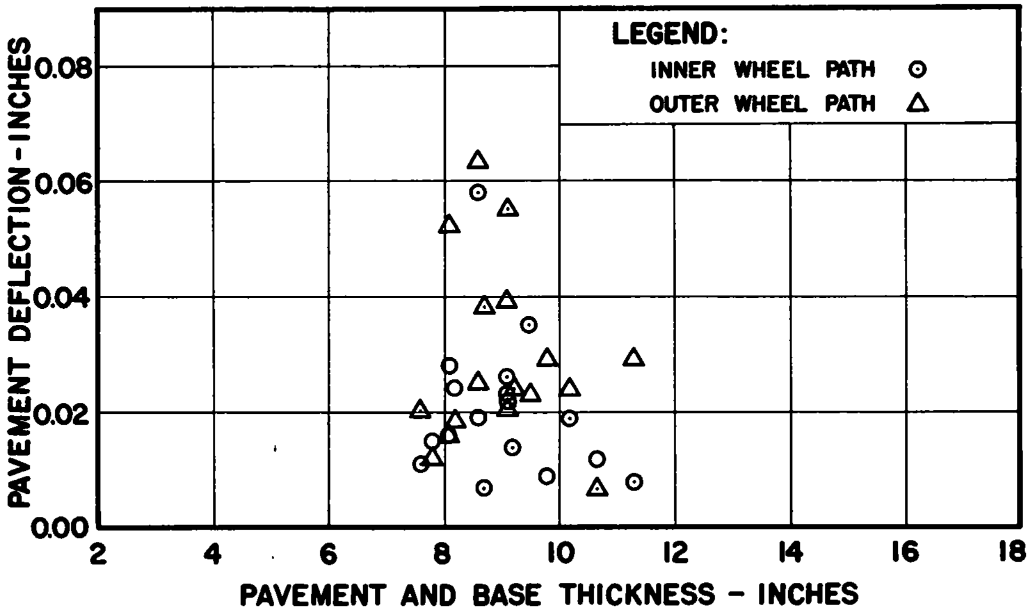

Figure 12. Deflection vs pavement structure, Job K.

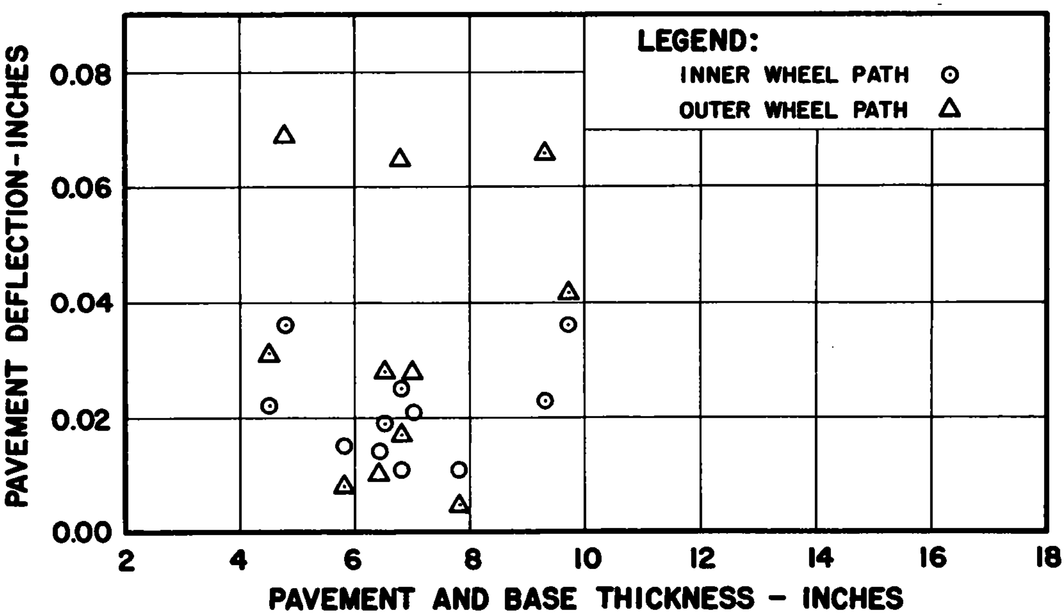

Figure 13. Deflection vs pavement structure, Job L.

Job K is a double surface treatment with moisture conditions very similar to the other jobs. The base material on Job K is of a better quality and is about 2 in. thicker. This project is in excellent condition and has required very little maintenance. The average deflections for the project are below the over-all average of the high-type pavements. Figure 11 shows that there is little variation in the deflection under the inside wheel from season to season.

Figures 12 and 13 show the variation of deflection with pavement structure. Job K (Fig. 12) is an example of an excellent double surface treatment in good condition. There is not enough variation in structure thickness to establish trends for this job. Job L (Fig. 13) is typical of the double surface treatment roads and the road mix Job E, also, in that the plotted points vary as if placed from a shot gun. No trend can be established. This job has narrow shoulders and is beginning to require extensive maintenance, especially in the outer wheelpath.

Job E is as an example of a road in very poor condition. The deflections listed are those occuring when a pavement requires extensive maintenance and could be considered a total failure.

Determination of the radius of influence shown by the graph from the Helmer recorder is not complete.

Condition survey data are shown in Table 4. The minimum condition ranges from 60 percent on Job E to 75 percent on Job K.

The poor condition of Job E is reflected in the condition survey. Job K is the surface treatment constructed on crushed rock base. The pavement does not show any signs of distress. The observations of this job indicate that a double surface treatment can show higher deflections than a high-type pavement and still be in good condition.

DEFLECTION RELATED TO PAVEMENT THICKNESS

A plot of pavement deflection along Job I for the outer wheel is shown in Figure 9. The average deflection from station 0 to station 36 is 0.044 in., and the average pavement thickness here is 1.8 in. The average deflection from station 37 to station 46 is 0.026 in. and the pavement thickness is 3.1 in. The average deflection decreased by 0.018 in., or about 41 percent, where the pavement thickness increased. Data for the inner wheel are deflection averaged 0.031 in. from station 0 to station 36, and 0.026 in. from station 37 to station 46. The average deflection decreased 0.05 in. or 16 percent with the increased pavement thickness. This decrease in deflection for both wheels is credited primarily to a double layer of hot-mix asphaltic concrete pavement encountered from station 37 to station 46.

SUMMARY

The pavement deflection in the inner wheelpath is more uniform than in the outer wheelpath and changes only slightly with the season.

The deflection in the outer wheelpath is normally greater than the inner wheelpath, averaging about 40 percent larger on the high-type pavements and about 45 percent larger on the low-type pavements.

On high-type pavements there is a definite trend that deflection is proportional to thickness of pavement structure.

CONCLUSION

The zone of influence for a wheel loan can be measured using a Benkelman beam with Helmer recorder. This is true only so long as this zone of influence does not reach the beam supports. The graph drawn by the Helmer recorder shows where the zone of influence reaches from the point of maximum deflection. When the initial deflection extends beyond the beam support, this condition is immediately shown by the trace of the deflected point deviating from a horizontal line.

The deflection of pavement alone is not sufficient information to indicate pavement performance. For example, Job F has an average deflection of 0.024 in. and is rated 70 percent condition, Job M has an average deflection of 0.020 in. and rates 98 percent condition, and Job I has an average deflection of 0.036 in. and rates 88 percent condition. Job M is the best pavement and has the lowest deflection, and Job I is a good pavement but has a higher deflection than Job F, which is a poor pavement.

The ratio of radius of influence to deflection can be used as a criteria for over-all pavement performance. For the high-type pavements studied a ratio radius to deflection for the outer wheel of 800 appears to divide the good from the poor pavements. Of the pavements reported, only Job I does not follow these criteria. The average ratio of Job I is 465, considerably lower than that indicating a good pavement; however, this pavement is classed as a good pavement with an average condition rating of 88 percent. Only future observations of this particular job will tell what the low ratio of radius to deflection actually means.

Flexible Pavement Design: A Complex Combination of Theory, Testing And Evaluation of Materials

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The purpose of this report is to point out certain complexities usually encountered in attempting to develop pavement thickness design methods. No attempt is made to offer specific answers, but rather to submit concepts that might assist in understanding the many problems involved in the use of methods for design of flexible pavements. The following recommendations are made:

1. Of the strictly empirical methods only those founded on an extensive background should be acceptable.
2. All empirical methods based partially on theory require some background before being acceptable.
3. Empiricism in methods should not predominate, but its presence does not indicate the method to be unworkable.
4. Each agency, if it has not already done so, should start to investigate at least one or more methods in lieu of waiting for someone else to work out the often referred to "rational approach." This report shows why the author believes that the establishment of a workable "rational method" is still a long way off, if at all possible.

A series of test questions is presented to assist the reader in evaluating any proposed method.

●FOR MANY YEARS to come acceptable improvements in flexible pavement design techniques for pavement thickness will be developed by a few organizations and sought after by many engineers. Many methods will be cast aside for various reasons. In some cases the reasons may be due to failure to follow the intent of the originator either through lack of understanding or through belief that the method is too cumbersome. Only a few organizations have the time, personnel, and facilities to develop methods covering such a complicated subject. The subject is so highly involved that all of the better methods to date have been developed over a period of many years of continuing research. In spite of various perferences and beliefs, there are and always will be those seeking the preferred methods, and it is hoped that this report will be of some benefit to such personnel.

All of the better techniques of design must be accompanied by adequate physical testing techniques that measure various relationships of compressive, shearing, and tensile strengths. Such tests must be supplemented with good judgment in design and application. Figure 1 shows some of the most commonly used test procedures. It can be noted that these tests measure one or more of compressive, shearing, or tensile strength characteristics. The idealized Mohr diagram shown in Figure 2 is one of the most helpful tools available for presenting the three mentioned stresses. Shearing stresses can be obtained when tensile and compressive stresses are known.

Figure 3 shows the existence of tensile and compressive stresses both under a tire load and in a reversed manner some distance away from the loaded area. This example merely illustrates how pavements are subjected to many repetitions of all forms of compressive and tensile stresses.

Figure 4 shows the basic concept of the Texas method where results of triaxial tests and wheel load stresses can both be presented on the Mohr diagram. However, to

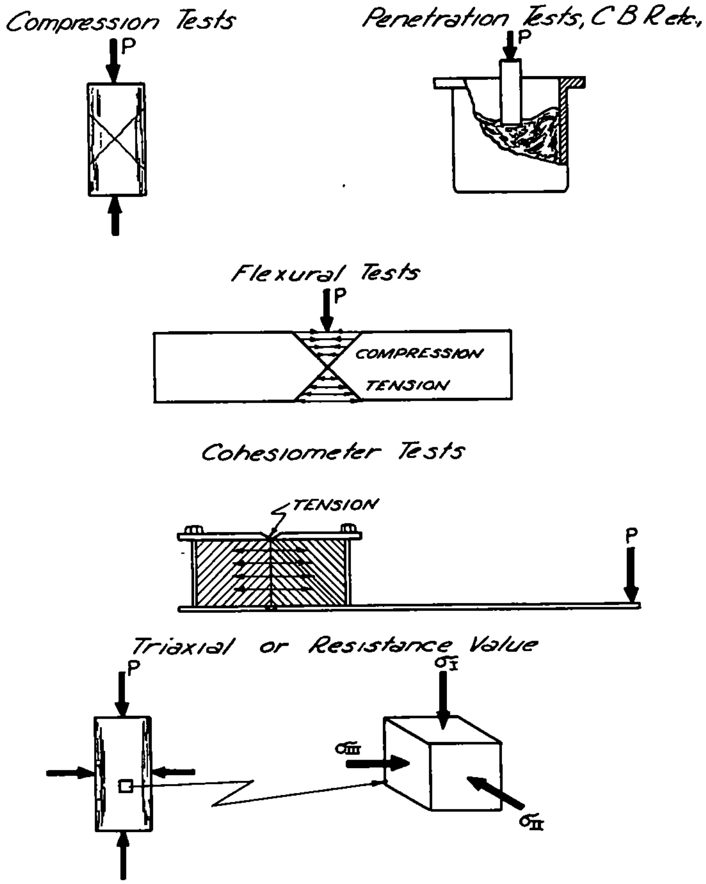

Figure 1. Tests most commonly used for pavement design.

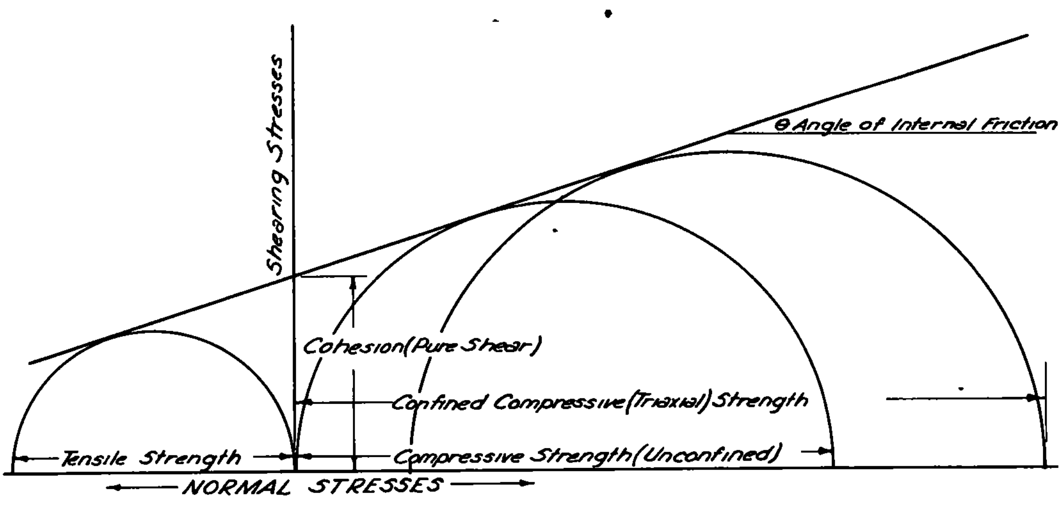

Figure 2. Idealized Mohr diagram.

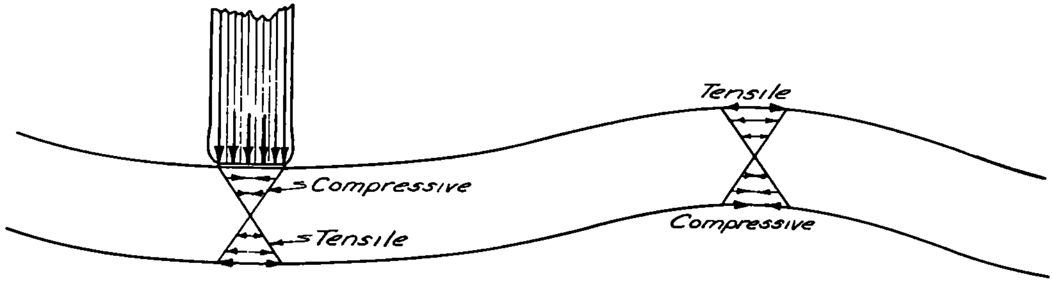

Figure 3. Stresses from wheel loads.

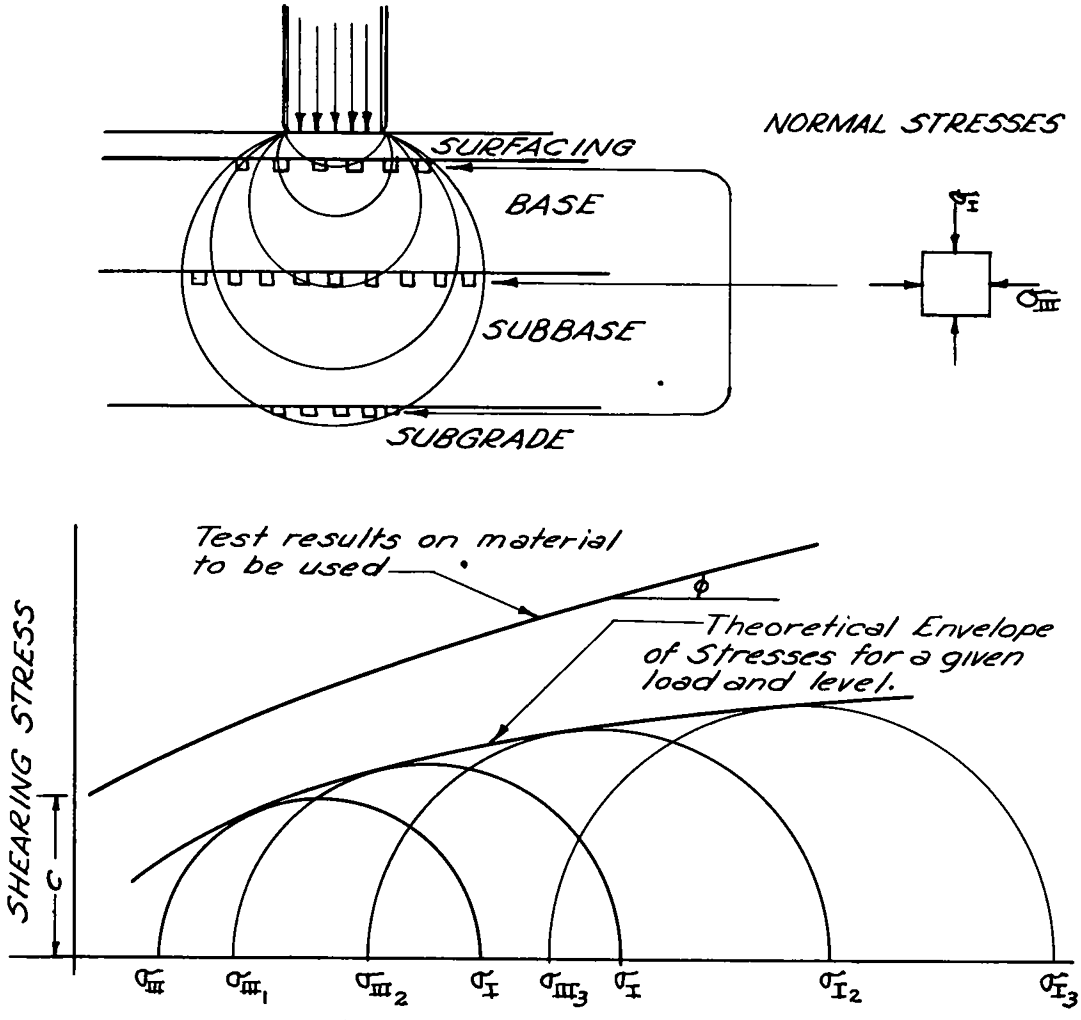

Figure 4. Comparison of wheel load stresses to triaxial strength test stresses.

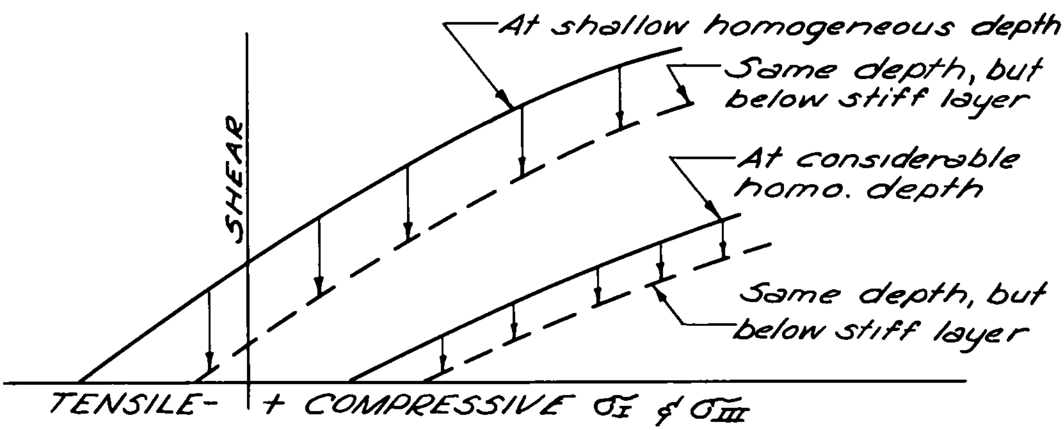

Figure 5. Stresses at different levels for a given loaded circular area.

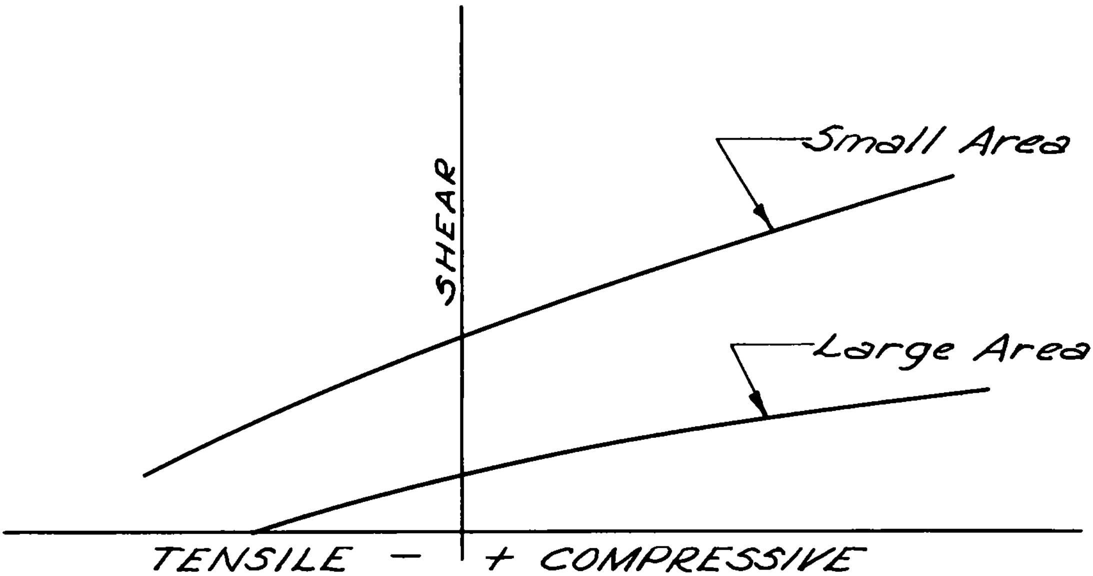

Figure 6. Stresses at a given depth level when loaded area and unit pressure varies.

present realistic values is not as easy as it first appears. The procedure usually followed is to utilize tables of influence values determined from Love's solution of Boussinesq's equations of elasticity for circular loaded areas. Because these values are for homogeneous isotropic layers, it soon becomes obvious that similar patterns of stresses can not be expected under a pavement system consisting of several layers of materials having different degrees of stiffness (1).

Figure 5 shows how stresses under a given circular loaded area will decrease with depth and will also be reduced when a stiff layer overlies a soft layer. To point out further complications, Figure 6 shows that for a given load the size of loaded area will affect stress concentrations at a given depth level. This matter is further complicated if consideration is given to adding braking and impact stresses not to mention stresses under moving wheel loads. Many complications also arise when a laboratory attempts to evaluate strength characteristics of materials.

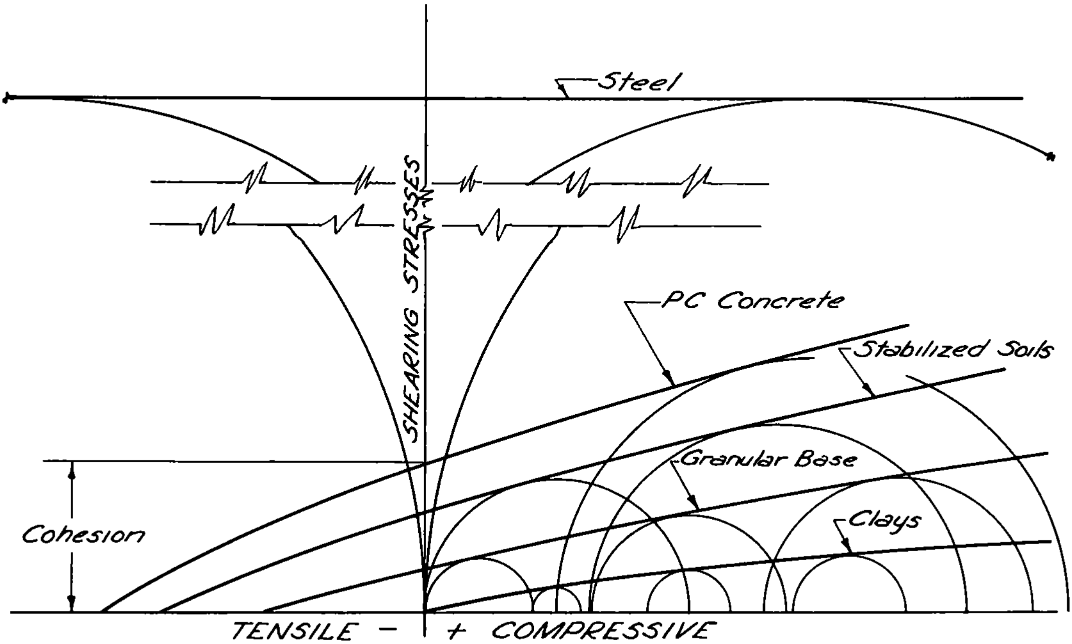

Figure 7. Triaxial test results for various typical materials.

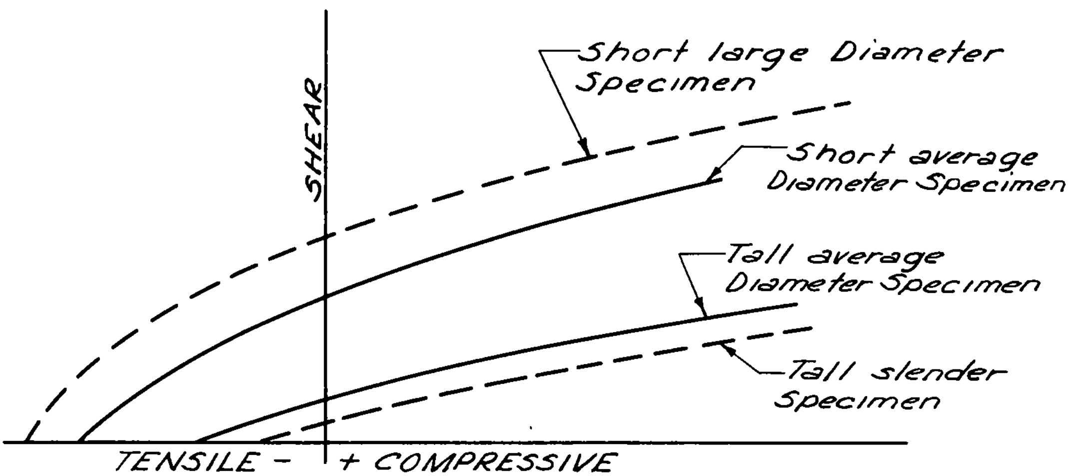

Figure 8. Effect of height of specimen on triaxial tests.

Figure 7 shows characteristics of various types of some materials when presented on the Mohr diagram. In order to present proper data, consideration should be given to a number of variables.

Figure 8 shows how height-to-diameter ratio affects strength results which are generally accepted. The lower dashed line in the figure suggests a condition not generally known, where specimens have a satisfactory height-to-diameter ratio that may

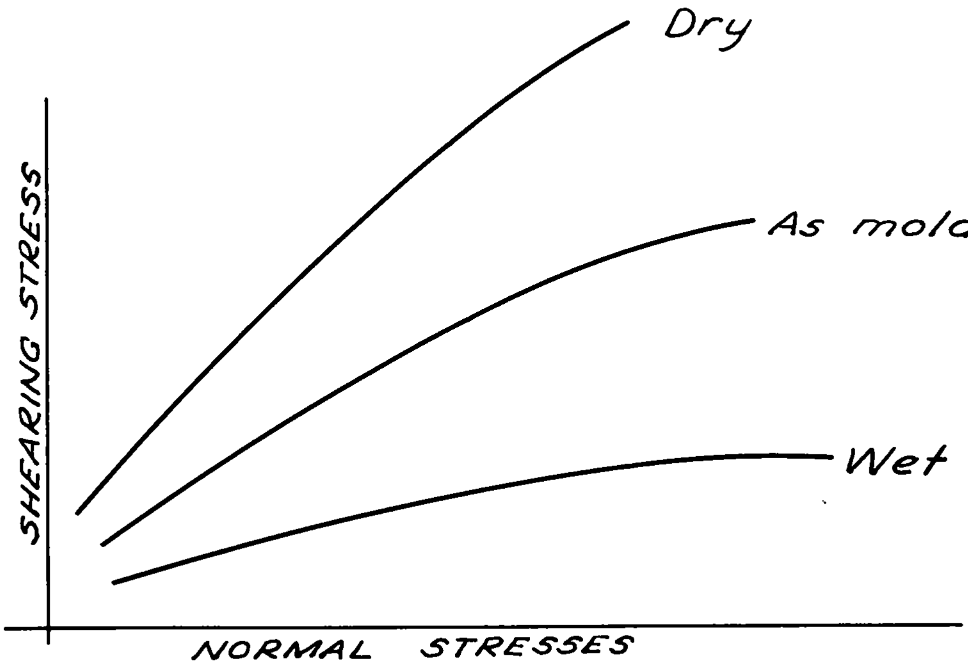

Figure 9. Effect of moisture on triaxial tests.

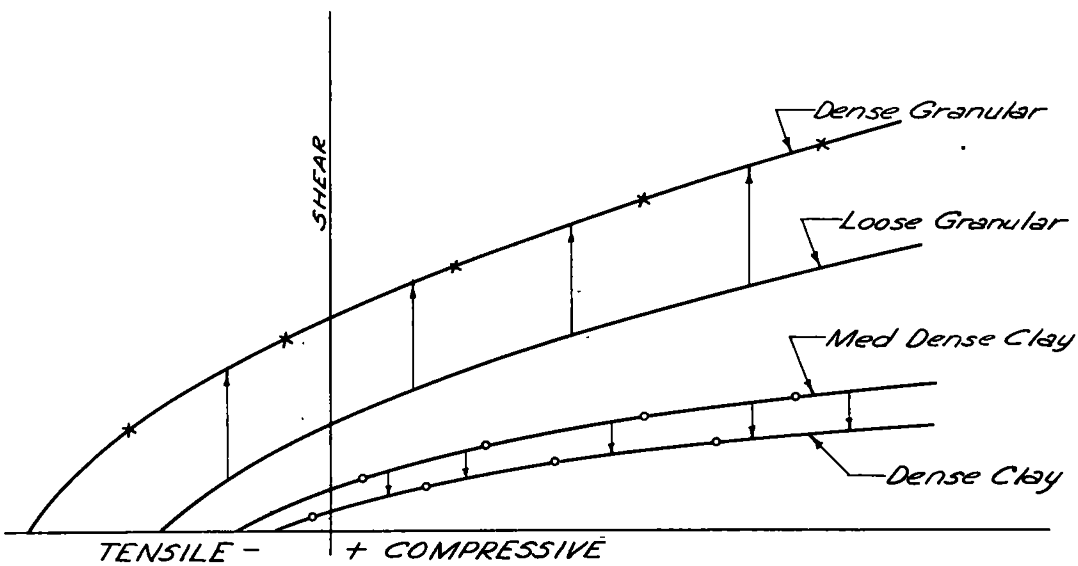

Figure 10. Effect of density on triaxial tests.

be too slender. In other words, for granular materials some 10- by 20-in. height specimens test stronger than some identical 6- by 12-in. height specimens. This suggests that specimens exceeding approximately 8 to 10 in. in height should have diameters in excess of 6 in.

If enough confusion does not already exist, the selection of moisture content of soil materials at time of testing should complicate things even more. As would be expected, strength varies with moisture content similar to the results in Figure 9. Selection of proper moisture content at time of testing is not a simple matter, and laboratory technicians have to select some degree of wetting at time of testing before they obtain acceptable data. To attain the desired moisture content for testing, some use capillary wetting (2, 3), some use inudation or soaking, and some use exudation plus soaking. All hope that the condition of moisture reached is comparable to severe conditions of the prototype, which may or may not be true.

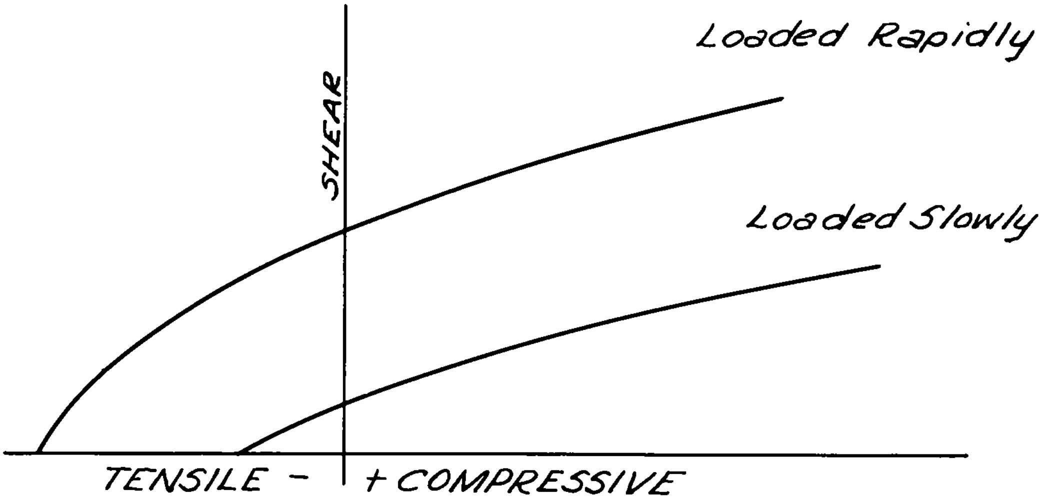

Figure 11. Effect of rate of loading on triaxial tests for natural or asphaltic bound mixtures.

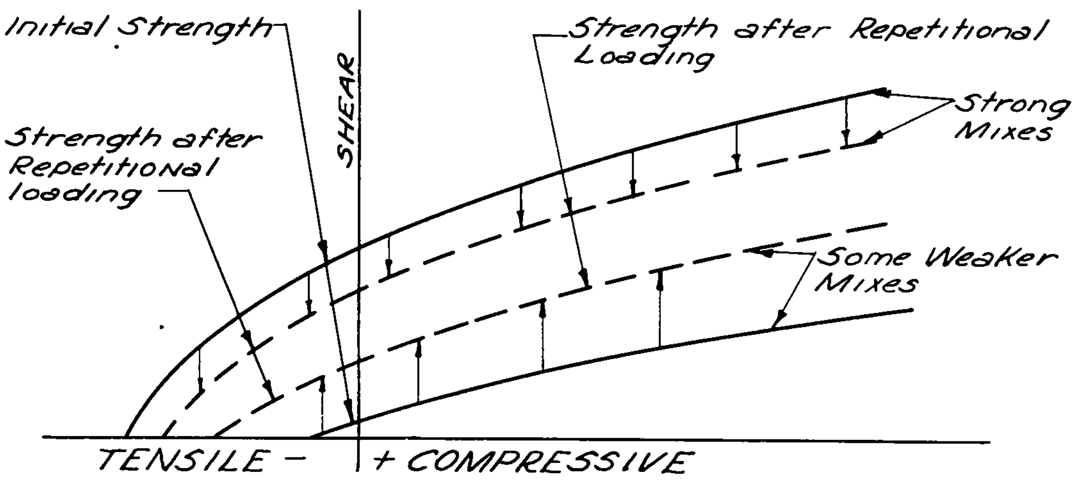

Figure 12. Effect of repetition of loads on triaxial tests.

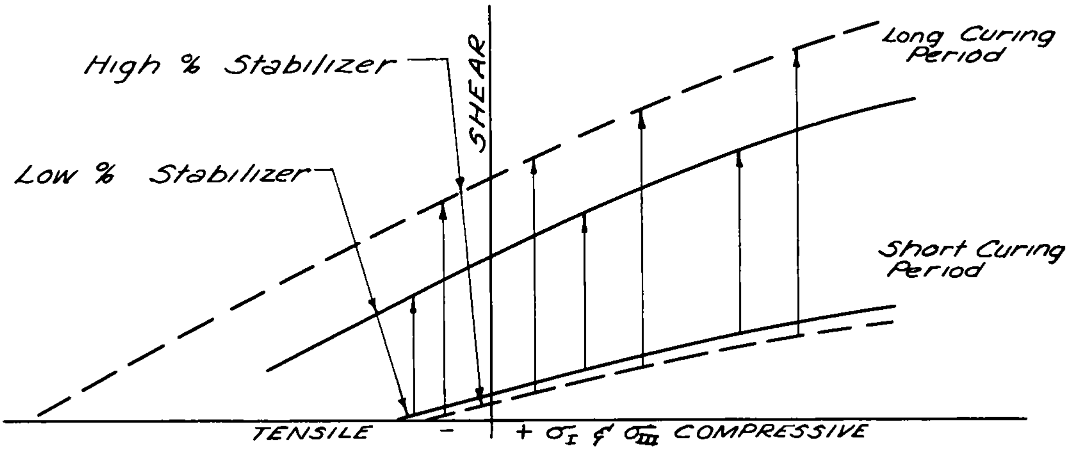

Figure 13. Curing of chemically cemented mixtures.

If prototype testing is followed, as desired by many, it is necessary to test specimens at densities comparable to road conditions (4, 5) because data will tend to vary, as shown in Figure 10. In the cases shown, a granular material gains in strength when accompanied by increased density; however, a clay may be stronger when compacted to a medium high density than when compacted to a higher density. Strengths referred to are after curing and capillary wetting.

Additional confusion arises when the rate of load applied during testing is considered. Figure 11 shows fast loadings on a given soil produce higher strengths than do slow rates of loading on the same soil. This may be obvious but the laboratory engineer must establish some standard rate for use in routine testing.

The next step to worry about is the effects of repetitional loading. Figure 12 shows some dense strong soils containing soft aggregates may weaken when subjected to repetitional loadings and some other weaker soils will gain in strength from repetitional loadings.

As if the pavement testing and design problems are not already complicated enough, there is the soil stabilization problem. Contrary to the expected, some stabilized mixtures containing high percentages of admixture may be weaker at early stages of curing but become much stronger after long periods of curing than do the mixtures containing low percentages of the same stabilizer, as shown in Figure 13.

Although contrary to most thinking, some cemented mixtures with fairly low compressive strengths may have greater tensile strengths than do some fairly high compressive strength mixtures (see Fig. 14). Soil and asphalt mixtures need to be tested to determine properties pertaining to absorption and shearing strengths of mixture selected for use.

Next to be considered is compatibility of materials; for instance, when two or more adjacent cemented layers having different linear coefficients of expansion and contraction tend to destroy each other, or when stable base materials feed excessive water to soils having little permeability. Then there is the wearing course or surfacing problem. It is suggested that the same type of surfacing should not be used on all types of designs. Most low-cost roads are designed for a limited life or number of load applications; therefore, deflections will be greater because base depths are thinner than if thicknesses were designed for long life. In these cases, penetration surface treatments will seal out water more effectively and last longer than premixes. This leads us to the complicated problem of selection of materials and application of surface treatments that have been the subject matter for numerous reports. If thick asphaltic concrete surfacings are to be used, an increase in aggregate size should accompany increasing thicknesses of surfacing. The need for stability tests and compaction by increased amounts of rolling to improve durability of such mixtures is becoming more

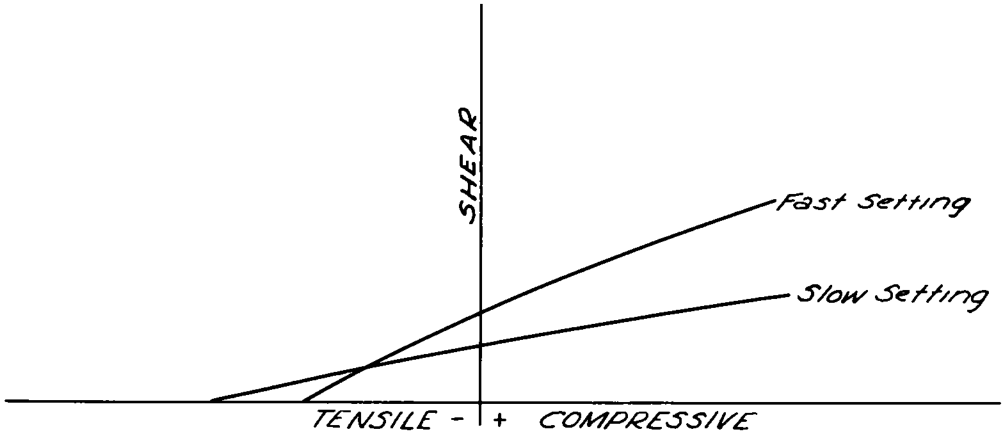

Figure 14. Curing of chemically cemented mixtures.

evident every day. This phase has also been subject matter for a voluminous number of reports.

Another factor often overlooked is the volume change of soils. Many pavements are badly cracked and their riding surfaces distorted regardless of thicknesses and types of pavements used without the builders or owners ever knowing the causes. This subject has been reported upon (3, 6) far less frequently than have other subjects discussed herein. This is unfortunate because many pavements in high volume change soil areas frequently develop cracks before traffic is ever allowed to use them.

This report may be thoroughly confusing with respect to the use of flexible pavement design methods, but that neither theory alone nor testing alone is sufficient within itself to furnish a means of coping with the design and construction of pavements. Good construction, drainage, evaluation of materials and their compatibility with other neighboring layers are also important, and these matters are not usually expressed in available theories and test methods. It seems rather remote to expect to find a truly rational approach to all of these problems. Through continued research, some methods can be correlated with field performance sufficiently for the user to expect good results in many cases, but he should study their application to his own particular problems.

In evaluating any method of design for flexible pavements, it is suggested that answers to the following questions be sought:

1. Does the method involve the use of theoretical wheel load shearing stresses from static plus impact loads?
2. Does the method account for wheel load repetitions or life of pavement?
3. Does the method evaluate the effects of tensile or flexural strength in certain portions of the pavements' structure?
4. Is there a sufficient background of actual experience?

No design method is any better than the test method that accompanies it and the following questions should be asked about the test method:

1. Are effects of both moisture and density registered in the test, and for routine testing are samples tested at conditions comparable to those of an adverse nature expected in the roadway?
2. Can aggregate-bearing samples up to 1 1/2- or 2-in. top size be tested so as to evaluate base and subbase materials?
3. Will the test indicate density desired during rolling?
4. Can test results be obtained within a reasonable time?
5. Can the test data be interpreted easily?
6. Can the amounts and affects of volumetric swell be measured?
7. Will the tests tell you how thick the surfacing should be and what characteristics it should have?

8. In the case of soil stabilization, will the test tell you the type of stabilizer and amount and thickness to use?

9. Are the test method results applicable to pedology and geology?

If all answers to these questions are yes, a mistake has been made, because no method is that good. If very few answers in the affirmative can be made about a proposed method, consideration should be given to use of other methods or to the fact that development of a considerable number of improvements through experience and research with one's own materials will be required to make the method work for you. One of the worst things that can be done is to adopt some method then change it so completely that its originator can no longer recognize it, because then it will have to be experimented with for 20 years before one can be sure whether it is any good or not. Perhaps it is even worse to do nothing while wishfully waiting for someone to come forth with rational approaches to all pavement problems.

Much has been done to improve the Texas triaxial method with respect to speed of testing, affect of load repetitions, and tensile-flexural strength on depths of pavement. Details relative to these matters are being given in another report (7).

CONCLUSIONS

From the voluminous data published on this subject, one cannot but help come to the conclusion that flexible pavement design is a complex combination of theory, testing, and evaluation of materials that has not and probably will never be resolved into a purely rational method. The enormous costs of pavements in current programs have created so great a need for good methods of design that use of the best methods obtainable must be started without waiting for the idealistic rational methods to be developed.

ACKNOWLEDGMENTS

The writer is indebted to many who have contributed and encouraged the development of this report. A few of these people or organizations have been mentioned in the text; however, it should be mentioned that the work of the members of the Soils Section and other members of the Materials and Tests Division of the Texas Highway Department, under the able guidance of A. W. Eatman, has been a major factor in making this report possible.

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Flexure of a Road Surfacing, Its Relation to Fatigue Cracking, and Factors Determining Its Severity

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This paper describes some aspects of theoretical and experimental work carried out in South Africa, over the past four years, in a study of "chicken net" cracking of a road surfacing, and of the flexure induced in it by wheel loads.

The accuracies of common measures of the severity of flexure are discussed. Of these, radius of curvature has been adopted for general testing. Results of field observations which reveal a distinct relation between curvature and degree of cracking of a surfacing are presented, a radius of about 125 ft being critical in the case of one road studied.

A field study is described which revealed that Elastic Theory is applicable to a limited extent in regard to the deflection and curvature of a road under its normal traffic.

A number of factors which affect the severity of the flexure developed beneath a wheel load are discussed. Curvatures are practically independent of the nature of the materials below a depth of about 15 in., and depend mainly on the materials of the base and subbase. Curvatures are affected considerably by the tire pressures of the vehicle inducing them, but little by the wheel load.

Existing design methods do not appear to take adequate account of flexure cracking, and the likely form of possible additional design considerations is discussed.

●AMONG THE possible causes of failure of a flexible road is "chicken net" cracking of the bituminous surfacing. Although in many cases it is not at first accompanied by loss of shape of the surface, such cracking can lead to failure, either by raveling and potholing of the surfacing, or by permitting rainwater to enter the foundations, with the possible consequence of loss of shape due to shear deformation. Cracking in this pattern may sometimes be attributed to internal changes in the bituminous surfacing. Work published by Hveem in 1955 (1), substantiating previous hypotheses, led to the now generally-accepted view that "chicken net" cracking may also represent fatigue failure under the repeated stresses in the surfacing which accompany the compression of the foundations and flexure of the surfacing under traffic wheel loads. It has not yet been established which of the stresses in the surfacing is the actual cause of the cracking, but it is likely that the tensile stresses at the points of greatest flexure play an important role (2, 3).

It appears that this type of failure is not adequately considered by normal methods of design, which are concerned mainly with the prevention of shear and consolidation failure in the foundations.

In about 1957, a problem arose in South Africa on the major road between Pietermaritzburg and Durban which, although designed generally in accordance with accepted CBR methods, and exhibiting no loss of shape of the surface, developed chicken net cracking in the thin premix surfacing over the major portion of its length. The heavy maintenance work undertaken proved to be of only temporary benefit, cracks often reappearing within a year. The National Institute for Road Research was invited to look into the reason for the cracking, and the investigation carried out gave the major impetus to the study of the flexure of road surfacings described in this paper.

A considerable amount of research is being carried out on flexure cracking in various parts of the world today. The work is concentrated mainly on studies of the flexibility of bituminous surfacings (2, 5), and on measurements of flexure in the field (5, 11). The emphasis in this Institute has been on the flexure developed beneath a wheel load in the field, and the various factors which affect its severity. The only similar study known to the author is that of Franck (12), which unfortunately only came to his attention during the preparation of this paper.

The first portion of the paper is devoted to a discussion of the various properties which may be used as a measure of the severity of flexure, and to field observations of the relations between two of these properties and the occurrence of cracking. Next, a study of the validity of applying Elastic Theory to a road structure is described, and the effects on the severity of flexure of various factors relating to the traffic and foundations are discussed. Finally, some aspects of a possible method of design to guard against flexure cracking are discussed briefly.

MEASUREMENT OF FLEXURE OF A ROAD SURFACING, AND SOME OBSERVATIONS OF CRACKING

If chicken net cracking represents fatigue failure of the surfacing under excessive stresses, then any study of the cracking should ideally include measurement of these stresses. The stress in a road surfacing would, however, be extremely difficult to measure. An indirect indication of its magnitude may be obtained by measuring the strains, provided the elastic moduli of the surfacing are known. In this regard, laboratory tests on bituminous mixtures by Saal and Pell (2), have suggested that strain may be more conveniently related to fatigue failure than stress, but more evidence is required before a conclusion such as this can be accepted, and stress must, for the present at least, continue to be regarded as the criterion for failure. Even strains, however, are difficult to measure in a road surfacing, and, except in special research investigations where such measurements have been carried out with success, some more simple measure is often desirable.

If an approximate measure is acceptable, then radius of curvature of the surfacing (or its reciprocal, curvature) may be used as an indication of the stresses. Curvature on its own cannot be an accurate measure, as other variables on which the stresses depend, such as the thickness of the surfacing, its Young's modulus (dependent on both its type and temperature), the Young's moduli of the foundations, and details of the traffic loading, all remain unspecified. The position can be improved slightly if reference is always made to the type, thickness, and temperature of the surfacing on which any curvature measurement is made.

The degree of approximation is further increased if only the maximum deflection beneath the wheels is measured. In addition to the other variables mentioned, that of horizontal dimension, or sharpness of the deflection pattern, now remains unspecified.

Although deflection would thus appear to be the least accurate of the four measures, and in fact to give no more than an approximate indication of the stresses, it is also the simplest to determine, and has come into widespread use. The theoretical indications that curvature would be a better measure appear to be recognized by many workers, but very few actually measure curvatures in practice.

Returning briefly to the neglect of the effects of stiffness and thickness of the surfacing when curvatures or deflections are measured, theory reveals a complex interrelation between these properties and the stresses, from which simple, generally applicable trends cannot be drawn. It is apparent, however, that for a given curvature or deflection, the flexural stresses are by no means constant but increase markedly with an increase in either the Young's modulus or the thickness of the surfacing. The trends between permissible deflection and thickness of surfacing given by Hveem (1), serve as an illustration of these effects. This dependence of stresses on the thickness and stiffness of the surfacing is discussed in more detail later in this paper.

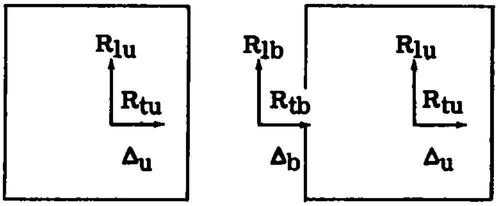
When deflections or curvatures are used as an indication of flexure, it is the general

TABLE 1

COMPARISON OF DEFLECTIONS AND CURVATURES UNDER AND BETWEEN DUAL WHEELS

SITE	$\frac{\Delta_u}{\Delta_b}$	$\frac{R_{lu}}{R_{lb}}$	$\frac{R_{tu}}{R_{lb}}$	$\frac{R_{tb}}{R_{lb}}$
A	1.19	0.85	0.73	-0.47
B	1.18	0.83	0.60	-0.47
C	1.07	0.81	0.72	-0.56
1	1.22	0.83	0.50	-0.38
2	1.13	0.83	0.49	-0.50
3	1.10	0.76	0.50	-0.52
4	1.08	0.93	0.75	-0.71

Symbols for deflections (Δ) and radii of curvature (R) induced by dual wheels



Test sites on Route 3/1 9,000- lb dual wheel load 75- psi inflation pressure 11 in. between centers of wheels.

practice to measure them between the dual wheels of a vehicle. The values so obtained are seldom, however, the highest occurring. This is apparent from the results of some tests carried out on a road in Natal (Fig. 1 and Table 1). The structure of this road was generally made up of a 1-to 2-in. premix surfacing, an 8-in. crushed rock base course, and a 6-in. silty-sand subbase, overlying a variety of natural materials. It is evident from the results that, for the conditions on this road, the deflections directly under the wheels, Δ_u, are higher than those between them, Δ_b, and also that the transverse radii of curvature, both between the wheels, R_{tb}, and under them, R_{tu}, are somewhat less (that is, more severe) than the longitudinal radius, R_{lb}, between the wheels normally measured. (These results seem to suggest that the transverse curvatures between the wheels are the ones most likely to cause cracking, because, in addition to the fact that these curvatures are the sharpest, the tensile stress occurs on the upper surface where weathering is likely to cause a deterioration in the bituminous material. The longitudinal elongation observed in many chicken net crack patterns is, as has been pointed out by others (3, 12), another indication that the factors giving rise to cracking are most severe in the transverse direction.) In addition, theory indicates that the relation between Δ_b and Δ_u, or between R_{lb} and either R_{tb} or R_{tu} is not a constant, but varies with the dimensions and spacing of the tire imprints and the relative elastic moduli and thicknesses of the layers of the structure. For example, in the case of a road where the Young's moduli of the surfacing and base are high, relative to those of the lower layers, a vehicle with closely-spaced tires may induce a completely different pattern of deflection, with Δ_b greater than Δ_u, and with R_{tb}, reversed in sign, less severe than R_{lb}. It becomes plain, therefore, that the deflection or curvature normally measured is neither equal to, nor constantly related to, the most severe occurring, and that yet another approximation is contained in the common procedure of using deflection or curvature as a measure of the stresses in the surfacing.

In the general testing work conducted in this Institute, a compromise between accuracy of representation of the stresses and simplicity of measurement has been adopted, and the longitudinal radius of curvature between dual wheels employed as the basic measure of flexure. A standard Benkelman beam is used for the measurement, deflections being recorded when a truck with dual rear wheels straddling the instrument stops

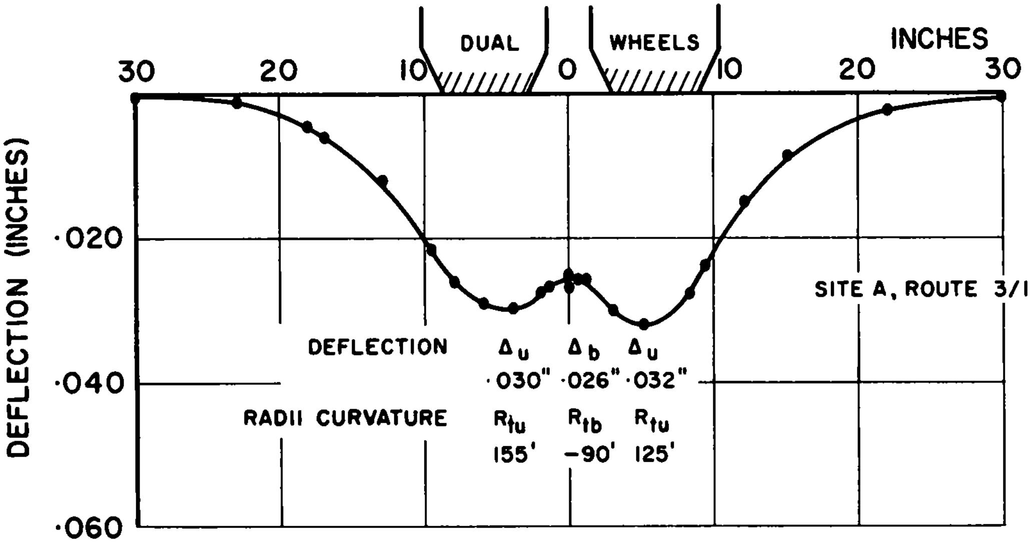


Figure 1. Typical example of transverse deflection pattern beneath dual wheels.

briefly every 6 in. of its travel over a distance of 4 ft on either side of the point of measurement. The longitudinal deflection pattern (actually an "influence" curve) is plotted from the results and the radius of curvature obtained by determining the radius of the circle of best fit to the plot at the point of maximum deflection (13). More recently this plot has been obtained directly on an X-Y recorder, using a Linear Variable Differential Transformer (LVDT) mounted on the Benkelman beam and a "Helipot" resistance attached to the wheel of the truck. It has been found that the repeatability of curvature measurements is not as high as that of the corresponding maximum deflection measurements, possibly because of the difficulty in stopping the truck at exactly 6-in. intervals. The accuracy has been improved with the introduction of the instrumented procedure, but as will be seen in curvature values quoted elsewhere in this paper, some scatter remains.

Deflections and radii of curvature have been measured on a number of roads in South Africa in varying conditions (11). An example of the results obtained on one of these is shown in Figure 2.

A distinct relation is evident between both the deflection and the radius of curvature developed under the test vehicle, and the condition of the surfacing. The fact that such relationships exist is taken as an indication (although not as a proof) that the chicken net cracking under consideration is in fact due to excessive flexures and, therefore, probably to excessive stresses in the surfacing — a confirmation of Hveem's findings. There is, however, a considerable amount of scatter in the plotted points. Probable reasons for this include the neglect, in these plots, of the effects of Young's modulus, thickness, fatigue strength, and age or traffic history of the surfacing, and the measurement of deflections and curvatures which were not the most severe occurring. It has not as yet been possible to isolate in the field the effects of any of these factors.

Further, it is seen from Figure 2 that for the 1- to 2- in. dense premix surfacings encountered on this road, and for the particular wheel loads and tire pressures used in the measurements, a deflection of about 0.055 in. and a radius of curvature of about 125 ft are critical, more severe values being associated with cracking. By comparison, critical deflections quoted by other workers for apparently similar surfacings vary between 0.025 and 0.050 in. (1, 6, 9, 12).

The existence of the relationships between deflection or curvature and the condition of the surfacing cannot be taken as evidence that both deflection and curvature are accurate measures of the stresses in the surfacing. In fact, these relationships merely indicate that in practice high stresses in the surfacing are, on the average, accompanied by high values of both deflection and curvature. Similarly, the fact that the deflection-condition relationship is almost as good as that for curvature cannot be taken as an indication that, contrary to the previous theoretical predictions, deflection is as good a measure of the stresses as curvature. Other effects could also give rise to a similarity between these two relationships. For example, although in individual cases deflection and curvature are by no means constantly related to one another, when results obtained on a variety of types of foundations under mixed traffic are considered

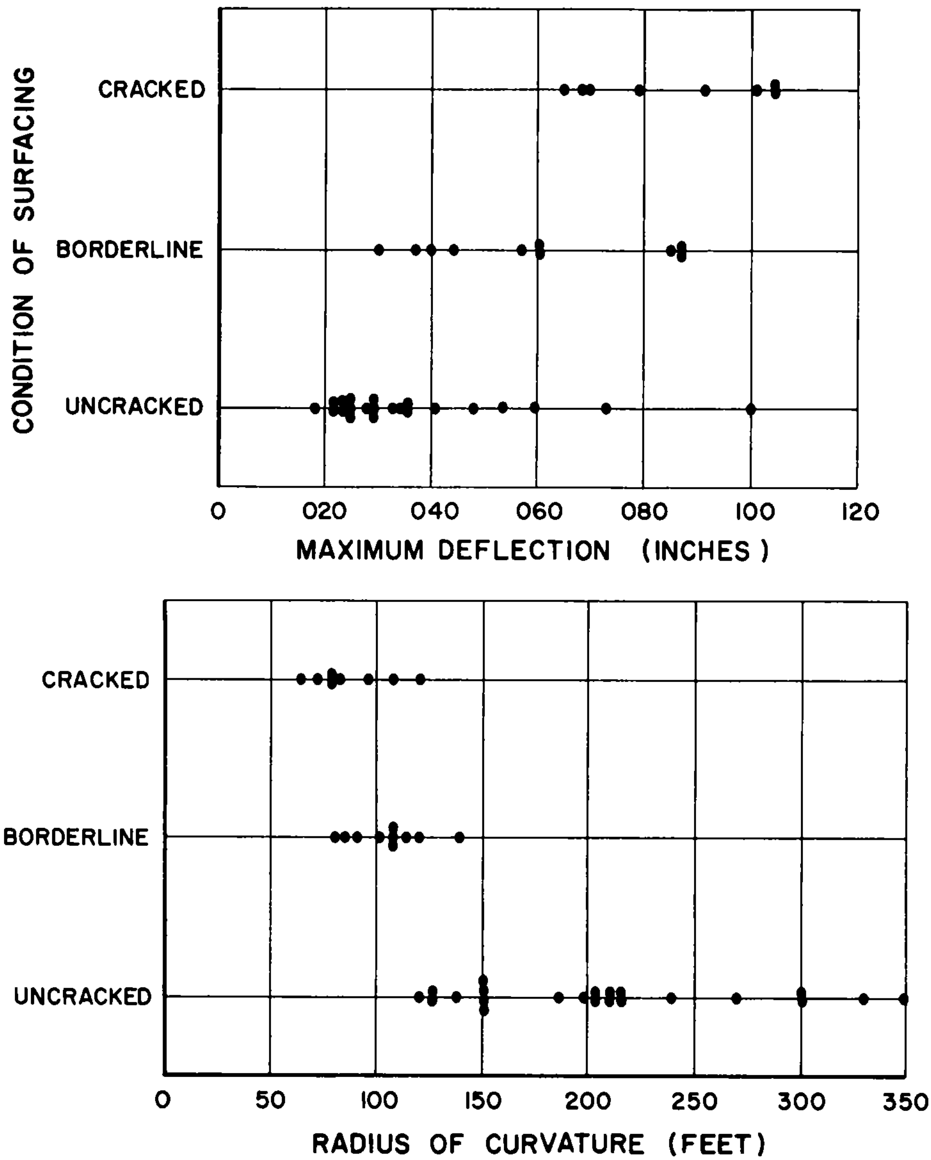


Figure 2. Relations between the degree of cracking of a surfacing, and the deflection and curvature induced by a dual wheel load (Route 3/1, 10,000-lb load at 90 psi, 1-to 2- in. premix surfacing).

together, on the average high deflections would be expected to accompany high curvatures and, therefore, apart from scatter, to be related to condition in a similar manner to curvature. Theoretical considerations thus remain the only indications at present as to the relative accuracies of deflection and curvature as measures of the stresses, and these favor curvature, as discussed previously.

Although deflection and curvature might be acceptable measures for general or routine testing, they are at best only approximately related to the stresses causing the cracking. For this reason, research using a more accurate measure, such as the strain in the surfacing, would be very desirable.

STUDY OF VALIDITY OF APPLYING ELASTIC THEORY

In much of the work conducted in studying the severity of flexure developed under a wheel load, Elastic Theory was employed at one stage or another. Use of this theory, however, was preceded by an investigation into the validity of applying it to a road structure, carried out on the major road referred to previously (14). This investigation is described in the following sections.

First, plate bearing tests of a specialized nature were conducted at twelve sites, to determine the Young's moduli of the layers of construction. Tests were carried out on the top of each layer, successive layers being removed by hand labor over an area of 12 ft by 12 ft. Inasmuch as the only concern was with those properties of the materials brought into play by normal wheel loads, and not with any consolidation or plastic shear, an attempt was made to limit the stresses applied to the same order as those to which the road had been subjected by traffic during its life. To this end, the mean pressures to be applied by the plates at various depths were selected in accordance with approximate calculations, based on Elastic Theory, of the stresses under a 9,000-lb wheel load at 75- psi tire pressure. Because the assumption of homogeneity within the layers would probably not be fully met, an attempt was made to simulate the traffic further by increasing the plate size with depth, from 12 in. on the surface to 24 in. on the subgrade. The loads were applied as rapidly as possible, and three or four repetitions employed to reduce errors due to initial bedding of the plate. When the resulting curves were plotted, it was observed that the load-deflection relationship in the last cycle was practically linear, indicating that the assumption of linearity on which Elastic Theory is based is fairly well justified in a road subjected to stresses of the same order as those applied by its normal traffic. This was not the case, however, when higher stresses were applied. A pronounced hysteresis in the unloading cycle indicated that Elastic Theory would be less valid for decreasing loads. Very little permanent deformation occurred after the third cycle, indicating that the assumption of elasticity was also justified. Young's moduli of the subgrades were computed from the slope of the loading curve on the last cycle using Boussinesq theory, assuming a Poisson's ratio of 0.5, and those of the subbases computed using Burmister's two-layer theory (15). (This theory unfortunately applies to a flexible loaded area, and in using it for the case of a rigid plate, as other workers had done, an error of unknown, but possibly considerable magnitude was introduced.) Because the surfacings were thin, the base and surfacing were taken together, and a combined modulus obtained using Odemark's approximate solution for a multi-layered system (16).

Second, before the excavations at these sites were begun, deflections and curvatures of the surface were measured between dual wheels in the normal way. At five of the sites, deflections and curvatures were also measured at various depths beneath the surface using a specially developed instrument fitting into a 1 1/8- in. diameter borehole. Deflections picked up at the desired depth were measured by means of an LVDT, which was referenced to a point 6 ft below the surface (10 ft in later tests). In order to have more simple loading conditions, single wheels were fitted to the truck, which then passed directly over the point of measurement.

With these two independent sets of observations it was possible to obtain a check on the validity of applying Elastic Theory. Deflections and curvatures of the surface and deflections at depth were computed from the Young's moduli measured, using Odemark's theory, and compared with those observed under the truck. (The fact that the Young's moduli used in these computations were themselves based on Elastic Theory was unfortunate, as it was later found that the isolation of reasons for discrepancies between observed and computed values was made more difficult.) Because of the doubtful validity of Odemark's theory away from the axis of load, no attempt was made to compute deflections or curvatures between dual wheels and, in the case of such tests, computations were made simply for a single circular load of the same area as the dual wheel imprint. The results are shown in Figures 3 and 4.

First, it is seen that there is a fair relation between the observed and computed deflections in Figure 3a. The scatter is probably due, in part, to experimental errors. In the tests with dual wheels, it is also probably due to the fact that the deflections beneath a single loaded area and those at a point between dual loads are not constantly related to one another, as discussed previously. A comparison between the absolute values can be made in the case of the tests with single wheels, and a major discrepancy is evident here in that computed deflections are some 30 percent less than those observed. No experimental error likely to cause a difference of this magnitude is apparent, and the discrepancy must therefore represent some limitation in the applicability of the theory.

The second test, the comparison of measured and computed curvatures, is more exacting, in that curvature is a more complex property than deflection, and it would be expected that any difference between theory and practice would be magnified. The relationship in Figure 3b is seen to be more scattered than that for deflections. Further, there is again a large difference between the absolute values, the radii of curvature computed beneath a single wheel being some 60 percent higher than those observed, for which no reason other than a limitation in the applicability of the theory is apparent. Viewed in another way, the deflections observed were more concentrated about the load than is indicated by theory. This was also the case at depth, the deflection pattern at subgrade level being more concentrated than even the Boussinesq theory for homogeneous

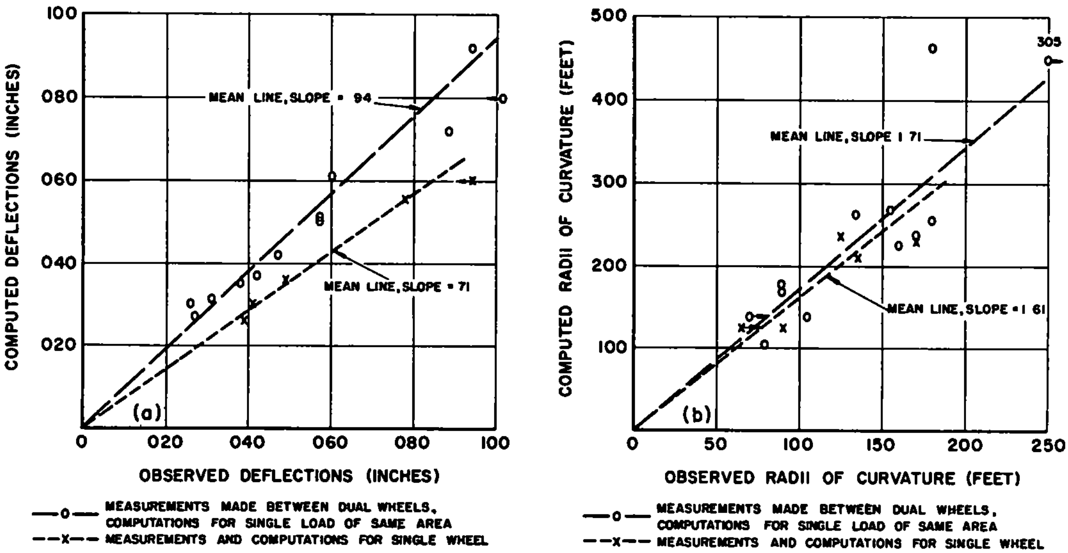


Figure 3. (a) Comparison of observed and computed deflections (Route 3/1); (b) Comparison of observed and computed radii of curvature (Route 3/1).

materials indicates. A similar effect was noted in the Waterways Experiment Station test on homogeneous soils (17). It is possible in the case of a layered structure that this is due to the lack of tensile strength in a crushed rock base course. Tensile resistance is implicit in Elastic Theory, and if the high tensile stresses in the upper layers cannot be carried, there will be less "slab effect" and more concentration of the deflections than indicated by theory. An error such as this would also have affected the moduli determined from the plate tests. The fact that the two errors would tend to cancel one another suggests that the true error is even greater than the difference indicates. It is possible that the observed discrepancies might be reduced by the introduction of some form of concentration factor into the theory, as has been suggested previously for stresses.

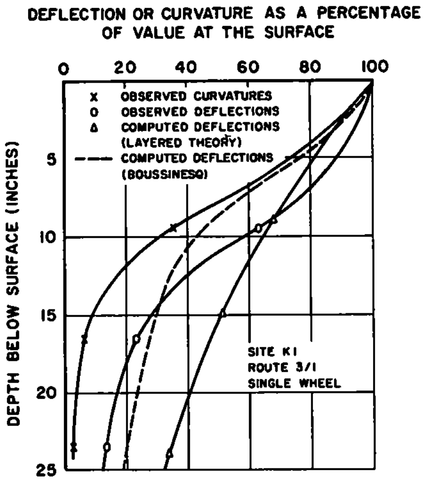


Figure 4. Comparison of observed and computed variation of deflection with depth.

The third check of the theory relates to the variation of the deflection with depth. A typical result is shown in Figure 4. The most striking fact, which recurs to a greater or lesser extent in all the tests, is that the reduction in observed deflection with depth was more rapid than that computed. This was most pronounced in the lower layers where in many cases, such as the one pictured here, deflections were less even than those computed from Boussinesq theory. Independent checks revealed that this was not an experimental error due to excessive movement at the depth of founding of the reference rod (6 or 10 ft); in fact, movements at this depth were also smaller than those predicted by theory. No other likely experimental error is apparent. The Waterways Experiment Station tests gave similar results.

The results of the study indicate that differences exist between Elastic Theory and practice, mainly in that deflections are more concentrated about the load than indicated by theory, and that they decrease more rapidly with depth. Although it would be of considerable value if the actual reasons for these differences could be established, this is not essential for the problem under consideration. It may be concluded from the results that Elastic Theory, for a given type of construction and under the normal traffic, may be used with fair confidence as far as relative values of deflections at the surface are concerned and also, although with a lesser degree of confidence, for relative values of curvature at the surface. Elastic Theory could therefore be used, for example, to indicate qualitatively a deficiency in any of the layers of construction, or to indicate general trends in the effects of certain variables on deflection or curvature.

FACTORS DETERMINING SEVERITY OF FLEXURE

In the course of the investigations into the road failures referred to previously, a considerable amount of attention was paid to possible reasons for the severe flexures which developed under traffic loads. Some of the findings of this study are as follows.

Factors Relating to Road Structure

As became evident in the previous section, deflection of a road structure under a wheel load is an elastic phenomenon, and the magnitudes of the deflection and curvature developed are dependent on the elastic moduli of the layers of construction, together with their thicknesses. Elastic Theory, however, reveals an important difference between deflection and curvature in this regard. This is given in Table 2, where the effect of a change in the Young's modulus of any one of the layers of construction on the deflection and curvature at the surface has been computed for a range of flexible road structures using Odemark's theory. It is seen that, while the value of the maximum

TABLE 2

DEPENDENCE OF DEFLECTIONS AND CURVATURES ON STIFFNESSES OF MATERIALS IN UPPER AND LOWER LAYERS OF FOUNDATIONS

Layer	Thickness[a]	Percentage Change in Deflection or Radius of Curvature at Surface Resulting from a 50 percent Change in the Young's Modulus of the Layer Indicated					
		Case 1[b]		Case 2[b]		Case 3[b]	
		Δ	R	Δ	R	Δ	R
Surf and base	8 in.	16	38	14	32	21	32
Subbase	8 in.	11	9	9	13	11	14
Subgrade	Semi-infinite	25	3	28	5	19	4

[a]Assumed a 12-in. diameter circular load-equivalent to 9, 000 lb at 75 psi.
[b]Computed for the following relative Young's moduli of the layers (base, subbase and subgrade): Case 1—10:2:1; Case 2—10:5:1; Case 3—4:2:1.

deflection is dependent to a large degree on the Young's moduli of the materials at depth (subgrade and below) in addition to those of the upper layers, the radius of curvature is dependent mainly on the moduli of the upper layers of construction, and very little on those of the materials at depth. The shortcomings found in Elastic Theory in the previous section would mean, if anything, that the effects of the materials at depth were even less. (If desired, the dependence of curvature mainly on the materials in the upper layers of construction may be visualized simply by considering the small area of the base which is stressed by the wheel, contributing a sharp depression and curvature, and the larger area of subgrade stressed by the spreading load, which contributes a more gentle deflection pattern.) Comparisons between deflection and curvature were also obtained experimentally in the subsurface tests referred to previously. A typical example of the results is shown in Figure 4. It is evident that the curvature developed in regions closer to the surface than did the deflections, substantiating the foregoing theoretical indications, qualitatively at least.

As might be expected from Table 2, Elastic Theory indicates that, although a very poor subgrade could give rise to curvatures at the surface of a severity normally associated with cracking, this would be an extreme case. If curvature is an adequate measure of the stresses giving rise to cracking, and a better measure than deflection, then for all practical purposes the nature of the materials below a depth of 15 or 18 in. would not appear to be of any importance as regards flexure cracking—a conclusion of considerable importance in regard to design, and one contrary to the present general belief that the "resilience" of the subgrade is the important factor.

The factor requiring the main consideration in design against flexure cracking, therefore, is that of the Young's moduli of the materials in the upper layers and particularly in the base. Common design specifications usually preclude the use of anything but a high-quality material in the base, and to a lesser extent in the subbase. It is, however, not as yet known whether these materials, although being of "high quality" in the general sense of the term, will always also possess a correspondingly high Young's modulus. There are, in fact, indications to the contrary, among which is the experience in Natal with certain subbase materials which were generally satisfactory as regards CBR, but unsatisfactory as regards Young's modulus (14).

Turning now to a second structural aspect, it appears that the laying of a thin (up to 1 1/2-in.) chip and spray or premix surfacing has very little effect on the deflections or curvatures developed under a wheel load. This is not the case with a thick (4-in.)

dense asphaltic concrete which, with its high Young's modulus and "slab" action, may have a considerable effect in reducing the flexures developed. For example, at a site near Pinetown in Natal, a 4-in. overlay of rolled asphalt to B.S. 594 reduced deflections from 0.057 to 0.028 in., and increased radii of curvature from 90 to 350 ft (14). Similar effects of asphaltic premixes in decreasing the pressures transmitted to the subgrade have been described by Whiffin (18).

In discussing surfacings, however, curvature does not give a sufficiently true indication of the flexural stresses and, the aforementioned advantages are offset to some extent by the general increase in stresses for a given curvature as the thickness or modulus of the surfacing increases. Turning to theory, it appears that for the case of a single wheel, stresses are a maximum in a surfacing of some intermediate thickness —for greater thicknesses the reduction in curvature due to the structural effect predominating over the tendency for the stresses to increase, and inversely for thinner surfacings (16, 19, 20). For common foundations, types of surfacings, and traffic, theory gives this critical thickness as being in the range 2 to 4 in. There is, however, not sufficient evidence at this stage for it to be suggested, for example, that intermediate thicknesses be avoided in design. Similarly, theory indicates that an increase in Young's modulus of a surfacing will generally result in an increase in the flexural stresses. An increase in Young's modulus, therefore, appears to be a disadvantage as regards flexure cracking although, in practice, depending on the design of the surfacing, this is often offset by an accompanying increase in the fatigue strength (3).

Factors Relating to Traffic

In addition to being dependent on the properties of the foundations, the flexure developed under a wheel load is affected by various factors relating to the traffic.

The effects of wheel load and tire inflation pressure on the flexure developed were studied at six of the sites referred to previously, by varying each independently in a series of tests. The surface deflection pattern was measured with the subsurface instrument, one of the dual wheels passing directly over the point of measurement. Inflation pressure was selected as one of the variables in preference to the contact pressure between the tire and the road surface (from which it differs considerably), because although contact pressure was the more fundamental property and of interest to the research worker, it was inflation pressure which was of greater interest to the practicing engineer. Typical results obtained are shown in Figure 5.

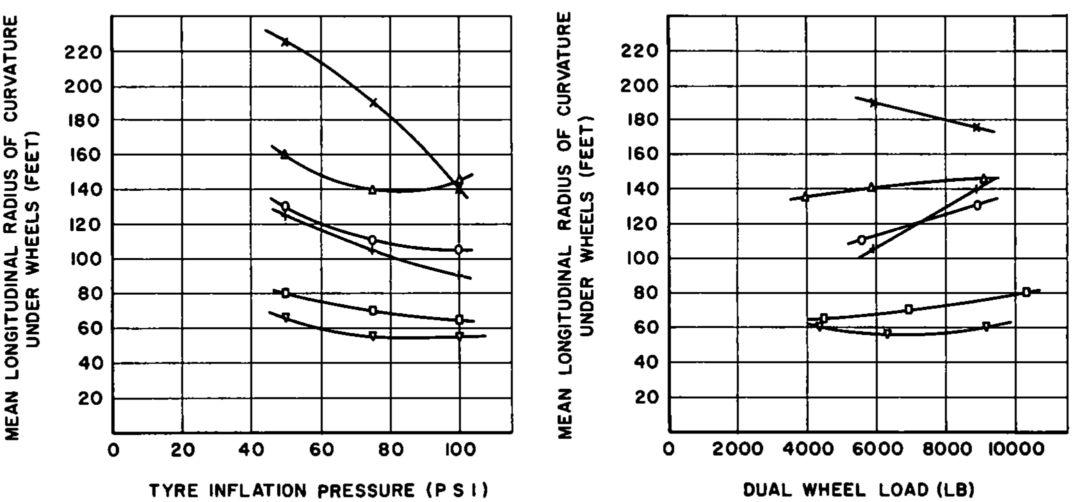


Figure 5. Variation of radius of curvature with wheel load and tire pressure at six sites (Route 3/1).

First, a marked trend is apparent in the plot of curvature against tire pressure. The scatter is probably due, in part, to experimental errors in the measurement of curvatures, as discussed previously. A simple relation between radius of curvature and inflation pressure fitting the mean results of all the tests over the range studied is $R \alpha\ p_i^{-0.5}$, whereas the corresponding relation in terms of the mean contact pressure is $R \alpha\ p_c^{-1.25}$. The pronounced decrease in radius of curvature may be visualized as being the product of two opposing effects. First, the decrease in size of the loaded area as pressure increased, giving rise to a more concentrated pattern of deflection and lower radius of curvature (the major effect); and second, the decrease in the effective depth of influence of the load with increasing pressure which, in the common case where the Young's moduli of the layers decreased with depth, would mean an increase in the effective stiffness of the construction, and hence an increase in the radius of curvature. The theoretical relation between radius of curvature and contact pressure for the case of a circular load on a homogeneous material is $R \alpha\ p_c^{-1.5}$. In view of the great number of variables involved, it is not possible to express a simple, generally-applicable theoretical relation for the case of a multi-layered system. A few simple deductions may, however, be made. According to theory, in the case where the moduli of the layers decrease with depth, the index n in $R \alpha\ p_c^{-n}$ would always be less than 1.5, and for common flexible road structures, excluding any with a thick bituminous surfacing or a cement-stabilized base, n would generally lie between about 0.75 and 1.5—a range within which the observed value of 1.25 falls. This pronounced effect of tire pressure on radius of curvature is a point of considerable importance in view of present-day tendencies to increase the tire pressures of vehicles, and one which would merit further study, both as described here, and with direct measurements of the strain in the surfacing.

Turning to the plot of curvature against wheel load, a considerable amount of scatter is evident, probably due again to experimental errors, which prevents the immediate recognition of any general trends. Analysis of all the results obtained, however, reveals a slight tendency for radius of curvature to increase (that is become less severe) with increasing wheel load at constant tire pressure, a relation of $R \alpha\ W^{0.25}$ fitting the results roughly. Although this may appear anomalous at first sight, it can be visualized, as before, as being due to the increase in the size of the contact area with increasing wheel load, giving rise to a less concentrated pattern of deflection and hence to a higher radius of curvature. The effect is, however, slight and the important indication given by the tests is that wheel load, as such, is not a contributor to severe curvature.

The tests conducted also enabled the variation of maximum deflection with pressure and wheel load to be studied. However, in view of the opinion that curvature is likely to be the better measure of the stresses in the surfacing, the deflection results are not of such great significance in the flexure cracking problem, and have not been presented in detail here. It may be of interest to note, however, that the mean relation between deflection and wheel load observed was $\Delta\ \alpha\ W^{0.8}$, while no effect of tire pressure on deflections was apparent.

Another factor which probably has an important effect on flexural stresses is the speed of the vehicle, but this has not, as yet, received serious attention in this Institute.

PREVENTION OF THE OCCURRENCE OF CRACKING

There are two main ways of preventing the development of flexure cracking. The first entails the provision of a surfacing with flexibility sufficiently high to accommodate the flexure encountered on the road. Studies of the performance of surfacings in the field, and research on methods of improving their flexibilities, are being made in a number of centers (2-11). The second method is to limit the deflections or curvatures which actually develop in the road, and it is on this aspect that work in this Institute has been concentrated.

As mentioned previously, there are indications that a road designed in accordance with normal methods, considering shear and consolidation, may not necessarily be satisfactory as regards the flexure induced by traffic. It therefore appears desirable that some additional design consideration be adopted which will insure low flexures. As discussed previously, the major factors relating to the foundations which affect the flexures developed are the Young's moduli of the base and subbase. The design consideration for flexure would thus ideally be based on the Young's moduli of these materials, for which there is at present, unfortunately, no accepted method of test either in the laboratory or in the field. Further, it appears that practically no knowledge exists at present on the Young's moduli of road materials and how they vary with the simpler indicator properties, such as grading, plasticity, and density. For this reason an investigation has been started to find a satisfactory procedure for measuring Young's modulus in the laboratory, and it is planned that a study then be made of how the moduli vary between different materials. Whether a knowledge of materials which possess low Young's moduli and should be avoided will be sufficient to prevent flexure failures in practice, or whether a complete method of design will be required, cannot yet be foretold.

CONCLUSIONS

The major findings in a study of the cracking of a road surface and the flexure induced in it by wheel loads, which has been in progress for some years now in South Africa, are as follows:

1. Either maximum deflection or radius of curvature of the surface may be used to give a rough indication of the stresses induced in a surfacing by a wheel load. Which of these is the better measure is not yet certain, although theoretical considerations favor radius of curvature. In the case of dual-wheeled vehicles, the deflections directly beneath the wheels may often be higher than that between them. Similarly, the transverse curvature, both between and under the wheels, may often be more severe than the longitudinal curvature normally measured.
2. A correlation exists between the condition of a surfacing and both the maximum deflection and the radius of curvature induced by a standard vehicle. The existence of these relations is taken as an indication that "chicken net" cracking is, in some cases at least, due to excessive flexure. For the thin premix surfacings studied, and for the particular test procedure employed, deflections of more than 0.055 in. and radii of curvature of less than about 125 ft are associated with cracking.
3. Young's moduli of the layers of construction of a road can be determined in the field using specialized plate bearing tests. On the basis of a field study on one road, Elastic Theory is applicable to a limited extent in a road structure under its normal traffic. It gives a fair indication of relative values of deflections induced by traffic at the surface, and a rough indication of relative values of the curvatures. Probably a large part of the reason for observed discrepancies is the assumption that the materials in the upper layers are able to resist tension, which, in practice, is often not true.
4. The radius of curvature of a road surface under a given vehicle is dependent mainly on the Young's moduli of the materials in the base and subbase, and if, as seems likely, curvature is an acceptable measure of flexural stresses, these become the important consideration in designing a road against flexure cracking, the subgrade being of relatively minor importance. The bituminous surfacing only has a significant effect in reducing deflections and curvatures if it is thick (more than 2 or 3 in.) and of a type having a high Young's modulus. Theory indicates that there may be a critical thickness of surfacing, in the range 2 to 4 in., for which flexural stresses are a maximum.
5. Radii of curvature are dependent to a considerable degree on the tire pressure of the vehicle inducing them (a series of tests on one road having yielded a relation $R \propto P_i^{-0.5}$), and only to a slight degree on the wheel load—conclusions of importance

as regards the relative detrimental effects of different vehicles in tending to cause flexure cracking. Deflections are dependent mainly on the wheel load ($\Delta \propto W^{0.8}$) and little on tire pressure.

6. There is considerable scope for further research on various aspects of the flexure cracking problem. At present, work in progress in a number of centers is aimed at improving the flexibility of surfacings; locally attention is being concentrated on reducing the flexure itself. Little knowledge exists on the Young's moduli of common base and subbase materials, and work is being done to find a satisfactory method of testing for these properties, to obtain an insight into the variables which affect them, and, should it prove necessary, to develop a method of design for limiting flexure, to accompany existing shear and consolidation failure design methods.

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Flexible Pavement Research in South Dakota

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In 1961 the South Dakota Department of Highways completed the second year of a three-year flexible pavement study. This study is intended to evaluate design procedures presently in use in South Dakota.

Thirty-four projects, totaling approximately 425 miles of highway, were selected as being representative of the various designs of highways currently being constructed. These test projects are being evaluated in terms of performance and cost. Performance is being determined through condition surveys, road roughness determinations, plate bearing tests, and beam deflection tests. Initial construction costs and annual maintenance costs during the life of the project are being recorded to compare with performance data.

Other factors which influence performance and cost are also being studied. These include the amount and type of traffic, type of soil, geology, and climate, and the quality of the various materials used in constructing the highway.

The results of this study will not be available until after the third year of testing. However, the procedures being used, the alterations made in procedures, the equipment being used, modifications made in the equipment, and some of the preliminary results of the study are discussed in this paper.

● IN 1961 the South Dakota Department of Highways, in cooperation with the Bureau of Public Roads, completed the second year of a comprehensive flexible pavement study. The main objective of the study is to evaluate the thickness design procedure presently in use.

Thirty-four projects, totaling approximately 425 miles of highway, were selected as being representative of the various designs of highways currently being constructed. The locations of these projects are shown in Figure 1. They are being evaluated principally in terms of performance and cost. Performance is being determined through condition surveys, road roughness determinations, plate bearing tests, and Benkelman beam deflection tests. Initial construction costs and annual maintenance cost data for each project are being compiled so that comparisons can be made with performance data.

Other factors that influence performance and cost are also being studied. These include the amount and type of traffic, type of soil, geology, climate and the quality of the various materials that are used in construction.

The results of this study will not be available until after the third year of testing; however, the procedures in use, the alterations that have been made in procedures, the equipment being used, modifications made in the equipment, and some of the preliminary results of the study are discussed in this paper.

CURRENT DESIGN METHOD

In 1946 the South Dakota Department of Highways started organizing a soil laboratory. One purpose of the laboratory was to study and evaluate the soils of the state and arrive at a thickness design procedure for flexible pavement most suitable to the climate, traffic, and other pertinent conditions found within the state. By 1952 the soil laboratory, after experimenting with several design procedures, had selected one that seemed to produce reasonable results. The design procedure adopted in 1952 is based on the CBR curves used by the Wyoming Department of Highways (1).

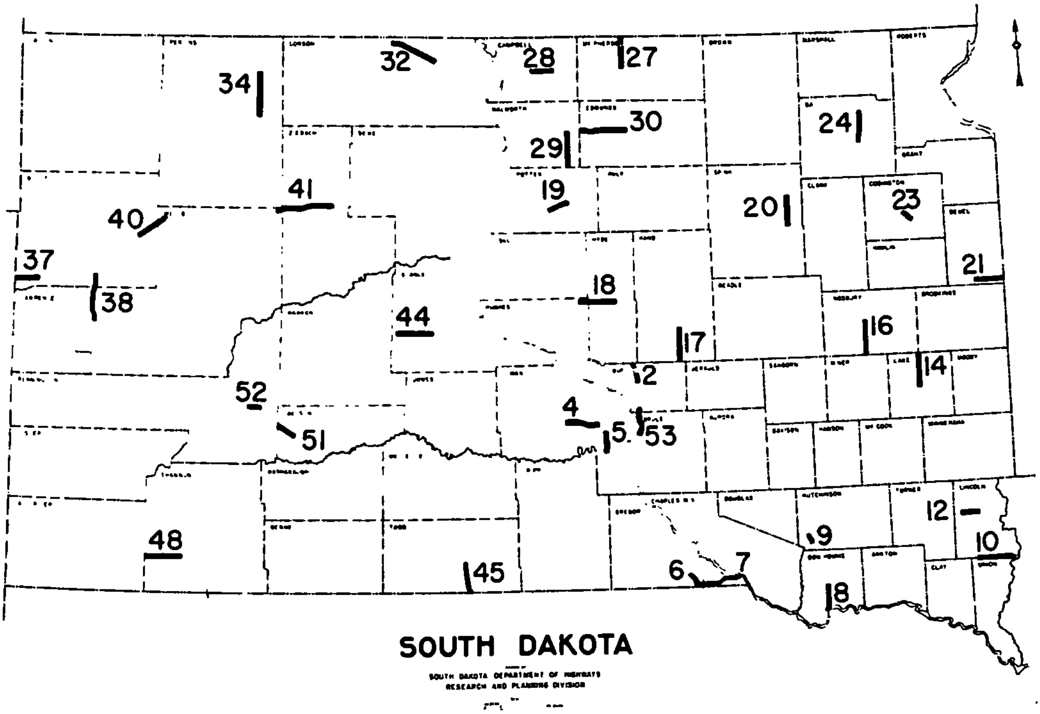

Figure 1. Project location.

In adapting this procedure for use in South Dakota certain modifications have been made. The most important change is the use of soil CBR values that have been determined from the liquid limit value of the soil. The soil laboratory personnel in South Dakota feel that there is a definite correlation between the liquid limit and the laboratory CBR of South Dakota soils. For this reason the bearing value is determined from a curve obtained by relating liquid limit and laboratory CBR values of many different soil samples.

SELECTION OF TEST SECTIONS

In selecting sections of highway for this flexible pavement research study it was, of course, desirable that they be as nearly representative as possible of the major highways within the state. The first requisite for a test section was that it be designed by the current design procedure. This meant that only roads designed and constructed between 1952 and 1959 could be selected.

Another criterion for selection of the test sections was that they be located within one of the six major soil association areas. South Dakota has been divided into 12 soil association areas, but the six major areas cover about 87 percent of the area of the state. This criterion was considered necessary in order that an attempt could be made to correlate highway design and performance with pedological soil mapping.

Three main types of bituminous surfaces are constructed in South Dakota. These are the class F or central plant mix using asphalt cement, class B or central plant mix using cutback asphalt and class A or surface treatment. These three types of pavement surfacing have been used on 80 percent of all the flexible pavement placed in the state and were being used exclusively at the time this project was originated. Class F and B pavements have been used on most of the primary system. Consequently these are the two pavement types with which this study is primarily concerned. However, two class A projects in one soil association area are also included. In general, three projects of each of the two types of surfacing, F and B, in each of the six soil association areas were selected.

SOIL AND GEOLOGY, TRAFFIC, AND CLIMATE

After the test sections were selected, soil and geological strip maps were prepared for each project. Where published maps were available the mapping was field checked and transferred to strip maps. However, most projects are located in unmapped areas. The South Dakota Geological Survey aided in preparing geological strip maps. Agricultural soil maps in sufficient detail are not available for most of the counties in the state, so a soil scientist was hired to prepare the soil maps. The correlation between projects by soil series names is not possible because so few areas have been previously mapped. However, the various projects were mapped and each soil assigned an identification number for use until such time as soil series names are established.

Traffic data have been recorded by the planning survey for each project during its entire life. Traffic in South Dakota is not so great a factor as in the more populous states. The most heavily travelled section had 1764 vehicles per day, 242 of which were commercial. The test section with the lightest traffic had 131 vehicles per day with only 18 commercial vehicles.

Other traffic studies made by the planning survey show that approximately 3.4 percent of all truck axles are loaded in excess of the 18,000-lb legal limit. Tire pressure studies indicate that 5.1 percent of all truck tires have air pressures over 90 psi.

Climatic data was obtained from the records of the U. S. Weather Bureau. The records from the weather station nearest each test section were used to obtain the temperature and precipitation data for that test section. The coldest wet winter between the years of 1948 and 1957 was determined and the cumulative degree days were calculated from the temperature records of that year. From the cumulative degree days the maximum depth of frost penetration was computed. In South Dakota it was found that the frost penetration ranged from 38 in. in the southern part of the state to 74 in. in the northern part during the period studied. The average annual precipitation ranges from 11 in. in the northwest to 25 in. in the southeast. The detailed weather records will be compared with pavement condition and maintenance costs to determine the effects that climate has on the flexible pavement in South Dakota.

MATERIAL CHARACTERISTICS

The materials used in constructing a highway have a definite influence on the serviceability of the road. To assist in the evaluation of the physical properties of the asphaltic materials and the aggregates used in the construction of each test section, the test results obtained during the design and construction phases were tabulated. In addition, samples of the surfacing, base course, subbase, and subgrade are obtained from each test section each year. The surfacing samples are approximately 1-ft square sections obtained when the mat is removed prior to conducting the plate loading test on the base course. The base course, subbase, and subgrade samples consist of approximately 100 lb of disturbed material obtained from the hole in which the plate loading test is conducted on the specific material. Sampling in this manner makes it possible to test in the laboratory the same material that was tested in place in the field.

The laboratory tests conducted on samples of the surfacing include asphalt content, asphalt penetration, and aggregate gradation. The base course, subbase, and subgrade samples are used to determine the gradation, liquid limit, plastic limit, wear loss, standard density, optimum moisture, and laboratory CBR.

In addition to these samples several 4-in. diameter cores of the surfacing are obtained for density determinations. Two-in. diameter undisturbed samples of the subgrade materials are obtained and the unconfined compressive strength is determined in the laboratory.

CONDITION SURVEYS

The condition survey method used was patterned after a method developed by R. A.

Helmer (2). The purpose of this type of condition survey is to determine the present structural condition of the road.

For clarity, the condition survey must be distinguished from a sufficiency rating, a roughness survey, or other types of road surveys or ratings. Condition surveys are limited to consideration of the structural condition of the road only. Roughness measurements obtained with one of the various types of roughometers provide a measure of the relative smoothness of a road. Although the roughness test gives an indication of a structural condition, it does not give the causes of such conditions. Sufficiency ratings take into account the condition of the road, and also consider geometric design and other factors. They are primarily concerned with the ability of the road to permit the existing traffic to move from place to place safely and comfortably.

In making a condition survey, it is desirable to express the results numerically. One way of doing this is to drive over the road and visually evaluate its condition. Inasmuch as this requires an estimation founded on the judgment of the individual, the personal factor is a major consideration.

The condition survey procedure used for this study was prepared in an attempt to reduce the human element and thus minimize the difference in rating assigned any specific road section by different observers. The road is rated in sections of not more than 0.2 mi in length at a time. The factors evaluated include ravelling, cracking, instability, crumbling, bleeding, and inconvenience to traffic. Two men in an automobile drive slowly along the road stopping every 0.2 mi, or more frequently if necessary, to examine the surface in detail. The various defects are noted and an adjective rating is assigned that particular section of road depending on the extent of distress. Within each adjective rating group there is a range of numerical values. The numerical value assigned to a particular short section of road depends largely on the previous experience of the observer. The adjective ratings and their numerical ranges are excellent, 98-100; superior, 90-98; good, 80-90; average, 65-80; poor, 50-65 and failure, less than 50.

The results of condition surveys conducted on the test projects at six different times over a two-year period are given in Table 1. Some difficulty was experienced during the first condition survey in the spring of 1960. The ratings for projects 2, 27, 29, and 34 are probably too high, and the rating for project 19 too low. Other than these discrepancies very little trouble was experienced in obtaining reproducible results. Improvement in the condition of projects 6, 12, 14, 32, and 40 are attributable to chip seal coats placed over the original surface.

ROAD ROUGHNESS

The South Dakota road roughness indicator is of the single wheel Bureau of Public Roads type. When testing, it is towed behind a panel truck at 20 mph. A 5-mi warm-up run is made before testing starts. After this run the tire pressure and other factors that may influence the readings are checked and adjusted to standard. All road roughness determinations for this study are made in the inner wheelpath in order to minimize the effect of paved shoulder width.

Some difficulties with the equipment have been experienced. Control of voltage has been a serious problem. The convertor supplied with the equipment did not provide sufficient voltage under sustained use. A larger convertor with a voltage control will probably be added this year. Some of the wiring was too light and has been replaced. Suspension bearings were improperly installed and had to be replaced. The two single-leaf springs, though meeting the dimensional specifications, did not meet the performance specifications. Under a 300-lb load the springs deflect only 2 5/8 in. instead of the 3 in. specified. Four new springs ordered for the equipment exhibit about the same characteristics but will be altered this year.

Soon after the road roughness indicator was acquired it was taken to North Dakota where several checks were made with the North Dakota and the Bureau of Public Roads roughness indicators. It was found that the results produced by the South Dakota device paralleled the others but consistently gave results that were 25 to 30 in. per mi lower. The equipment still tends to produce excessively low readings but it is hoped the modified springs will produce results that will agree with the equipment of others.

TABLE 1

CONDITION SURVEY DATA

Research Project	Condition Survey Data (%)					
	1960			1961		
	Spring	Summer	Fall	Spring	Summer	Fall
2	93	75	77	73	73	73
4	89	84	77	75	75	79
5	96	89	86	85	83	87
6	68	96	95	92	92	90
7	93	91	89	87	86	86
8	93	91	89	87	89	88
9	100	98	98	98	97	98
10	98	96	95	94	95	93
12	94	92	92	93	93	96
14	63	71	94	87	80	72
16	96	95	94	94	93	92
17	96	94	93	91	91	90
18	95	97	96	90	91	90
19	72	90	87	82	80	83
20	98	98	98	97	96	96
21	95	93	92	91	91	91
23	100	99	99	98	98	98
24	86	89	85	83	81	80
27	88	73	72	72	72	72
28	94	93	91	90	92	90
29	85	71	70	72	71	71
30	96	98	98	97	96	97
32	74	75	72	70	68	84
34	91	77	81	81	85	77
37	98	95	95	95	96	93
38	97	94	94	94	95	90
40	91	78	77	84	95	95
41	94	97	94	92	90	88
44	99	97	95	95	95	94
45	96	88	87	91	92	84
48	96	95	95	94	93	89
51	99	99	99	99	97	95
52	96	95	94	94	94	90
53	98	96	96	95	96	93

Some of the results obtained to date with this equipment are given in Table 2. Eleven of the 34 projects under study have been chip sealed since roughness determinations were first made. The chip seal increased the roughness index on all 11 jobs an average of 24 in. per mi. These increases ranged from 14 in. per mi on project 9.

Another interesting trend noted in the data is that the majority of the roads are rougher in the fall than in the spring and summer. During the two years of testing, the measurements of the spring run have been largest on only 6 percent of the projects, and of the summer run on only 6 percent of the projects. The measurements of the fall runs were largest on 88 percent of the projects.

TABLE 2

ROAD ROUGHNESS

Research Project	Road Roughness (in./mi.) 1960 Spring	Summer	Fall	1961 Spring	Summer	Fall
2	78	80	54	86	92	110
4	74	62	76	73	69	87
5	69	63	74	75	74	97
6	82	104	119	107	105	120
7	77	75	93	89	88	103
8	67	78	87	87	87	100
9	64	47	54	54	51	88
10	61	62	78	76	74	90
12	60	66	79	72	74	103
14	80	84	109	109	118	132
16	43	48	66	61	69	78
17	65	69	84	77	73	91
18	46	75	78	84	81	92
19	96	107	110	96	104	121
20	50	50	86	48	63	75
21	75	75	86	80	85	95
23	60	67	74	64	66	80
24	81	83	95	82	88	104
27	129	119	123	110	119	133
28	93	106	117	96	108	119
29	146	146	151	149	147	166
30	47	62	66	58	61	90
32	134	121	129	125	131	145
34	88	89	96	86	93	106
37	64	67	65	67	72	78
38	81	81	87	85	85	94
40	71	64	74	66	89	88
41	65	86	92	82	83	97
44	56	74	78	72	59	72
45	90	91	104	95	112	113
48	64	79	86	80	87	98
51	-	73	70	68	70	71
52	63	68	77	73	74	83
53	53	50	66	55	60	77

PLATE LOADING TESTS

On each of the test projects a typical section of road approximately 250 ft in length was selected as a plate loading test area. This section is located on a major soil series represented in the soil association area and is typical of the design of pavement over the entire project. The arrangement of the test locations within the test area is shown in Figure 2. These locations are 14 ft apart so that a test conducted at one location will not influence the test conducted at the adjacent location.

Circular steel plates 1 in. thick and 9, 12, and 18 in. in diameter are used. Loads are applied to the plates by means of a hydraulic jack and spacers. The reaction load is provided by two trucks loaded with water and placed back to back with a 14-in. wide flange steel beam rigidly fixed between them. The plates, spacers, and jack are shown in position for testing the base course in Figure 3. The trucks are shown in Figure 4. Each truck is loaded by means of a self-contained pump operating through a power take-off arrangement. The rear axle of each truck can be loaded slightly in excess of 26, 000 lb, which provides a total reaction load of over 52, 000 lb.

Before loads are applied to the plates, the truck axles are tied to the truck frames in order to lock the springs. This is necessary to keep the vertical movement of the trucks within the lift range of the jack ram during testing. Figure 5 shows the devices used to tie the axles to the frame.

The bearing plate is seated on a thin layer of dry sand passing a No. 40 sieve and retained on a No. 200. All three plate sizes are used for tests on the surface whereas only the 18-in. diameter plate is used on the base course, subbase, and subgrade.

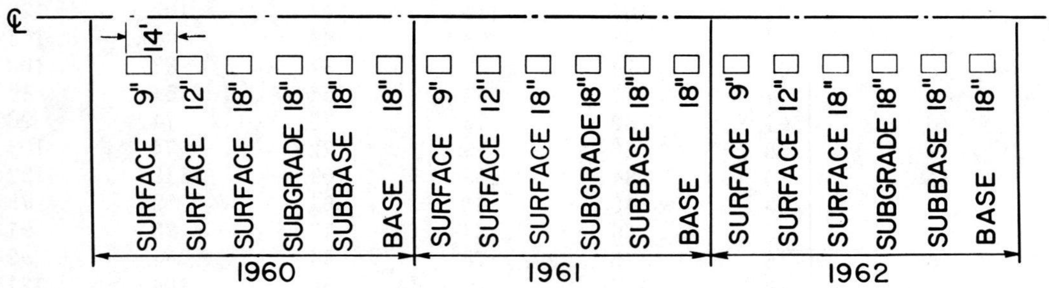

Figure 2. Typical project test site.

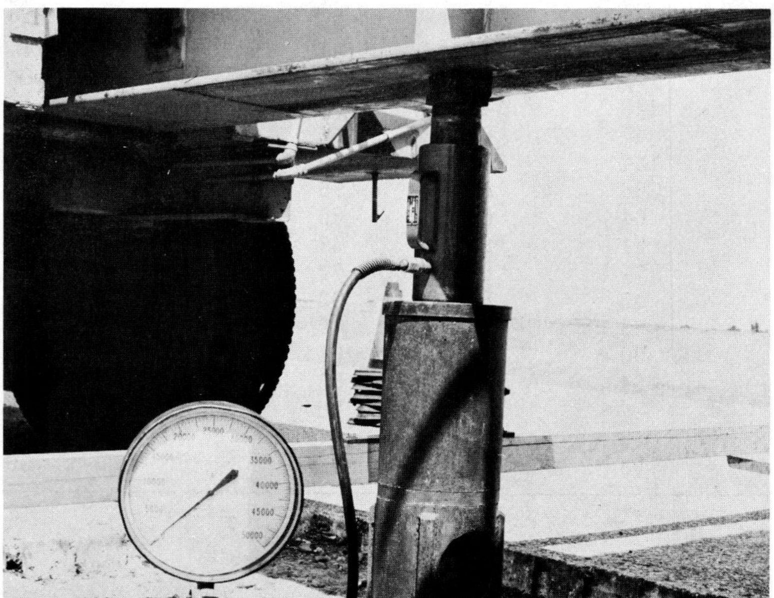

Figure 3. Plate load test equipment.

Figure 4. Plate load test trucks.

Figure 5. Truck axle tie-down.

Figures 6 and 7 show personnel digging the test holes in the pavement in order to test the underlying layers. A gasoline-type jackhammer is used to remove the mat and loosen the other layers for removal. This type of hammer is used because of its compact size and its eliminating the need for an air compressor.

The wide flange reaction beam and the aluminum datum beam are designed so that the center of the plate is 7 ft from the nearest truck tire and the datum beam supports are 7 ft from both the tires and the plate. This spacing is necessary to minimize the influence of pavement deflection beneath the tires on the area being tested and to provide a fixed datum to which the dial micrometer for measurement of the plate movement can be referenced.

The following definitions are included to facilitate understanding of the test procedures used:

1. Gross deflection. — The total vertical movement (elastic and inelastic) of the total pavement thickness, or combinations of the base course, subbase, and subgrade or of the subgrade only caused by the application of a single load or any number of loads.

2. Settlement. — The permanent or inelastic vertical displacement resulting from the application of a single load or the cumulative permanent displacement resulting from the application of a number of loads.

3. Elastic deflection. — The portion of the gross or single load deflection that is recovered upon release of the load.

Figure 6. Removing mat.

Figure 7. Loosening base course.

During the first year of plate load testing the incremental test procedure was used. A maximum gross deflection at the end of the test of 0. 20 in. was desired. After the surface to be tested was prepared and the plate seated under a load of 10 psi, a load was applied sufficient to produce 0. 05-in. gross deflection. This load was maintained until the rate of settlement reduced to 0. 001 in. per min or less. At this time an additional load was applied that was estimated to produce a gross deflection of 0. 10 in. This load was also maintained until the rate of movement was 0. 001 in. per min or less. Additional loads were than applied to produce gross deflections of 0. 15 and 0. 20 in., each load being held until the movement became 0. 001 in. per min or less. After the fourth, final load had been applied, all load was released and the rebound of the plate was measured when the rate of rebound reached 0. 001 in. per min or less. Testing was then discontinued at that particular location.

After the first year of testing it was decided to use a different procedure, the incremental-repetitional test procedure. It was felt that the elastic deflection of the medium tested might correlate better with the deflection data obtained with the Benkelman beams. The incremental-repetitional procedure makes it possible to differentiate between the elastic deflection and the permanent deflection (or settlement). As in the previously described method, a load was applied sufficient to produce 0. 05-in. settlement. This load was held constant until the rate of movement became 0. 001 in. or less in 15 sec. The load was then completely released until the rate of rebound became less than 0. 001 in. in 15 sec. The same load was then reapplied, held for 1 min, released for 1 min, reapplied for 1 min and finally released for 1 min. A gross deflection or a rebound reading was taken each time the load was applied or released for 1 min. This procedure provides three settlement readings and three rebound readings for each increment of load. The same procedure is followed for each of four load increments. As in the earlier procedure, loads estimated to cause 0. 05-, 0. 10-, 0. 15-, and 0. 20-in. gross deflection are used. After the last load increment has been applied for the third time the load is released and a final reading is taken when the rate of rebound becomes less than 0. 001 in. in 15 sec.

After the plate loading tests are completed, in-place California bearing ratio tests and in-place density and moisture tests are conducted on the base course, subbase, and subgrade. These tests are conducted on the same material and in the same hole in which the plate loading test was conducted on that particular pavement layer. The location of test areas in a test pit are shown in Figure 8. In addition, undisturbed sam-

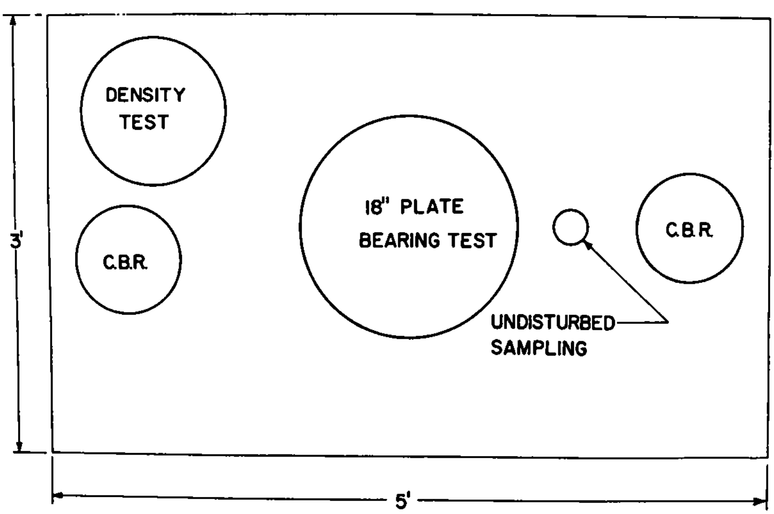

Figure 8. Arrangement of subbase tests.

ples of the subgrade material are obtained in the hole where the plate loading test is conducted on the subgrade. These samples are used for the determination of the unconfined compressive strength.

In addition to the field tests and laboratory tests described previously, the thickness of the various pavement components are also determined in the exposed test pits where plate loading tests are conducted on the subgrade.

BENKELMAN BEAM TESTING

For the purposes of this study the Benkelman beam with Helmar graphic recording attachment was desirable. In the early stages of testing, considerable difficulty was encountered with the beams and the linkage between the beams and the attachments. Figure 9 shows the original beam on the right and two modified beams on the left.

The six original Benkelman Beams were purchased in September 1959. The beams are of all aluminum construction, 14 ft in length, with a 12-ft long probe or lever arm. The probe is supported at the point of rotation by two ball bearings that are placed within the probe arm and turn on a shaft fixed to the frame of the datum beam. The fulcrum point of the probe arm is at the third point. This arrangement gives a 1 to 2 reading at the dial micrometer.

A standard 6-volt doorbell buzzer is attached to the side of the beam and operates from a lantern battery suspended under the beam. The buzzer is turned on during each

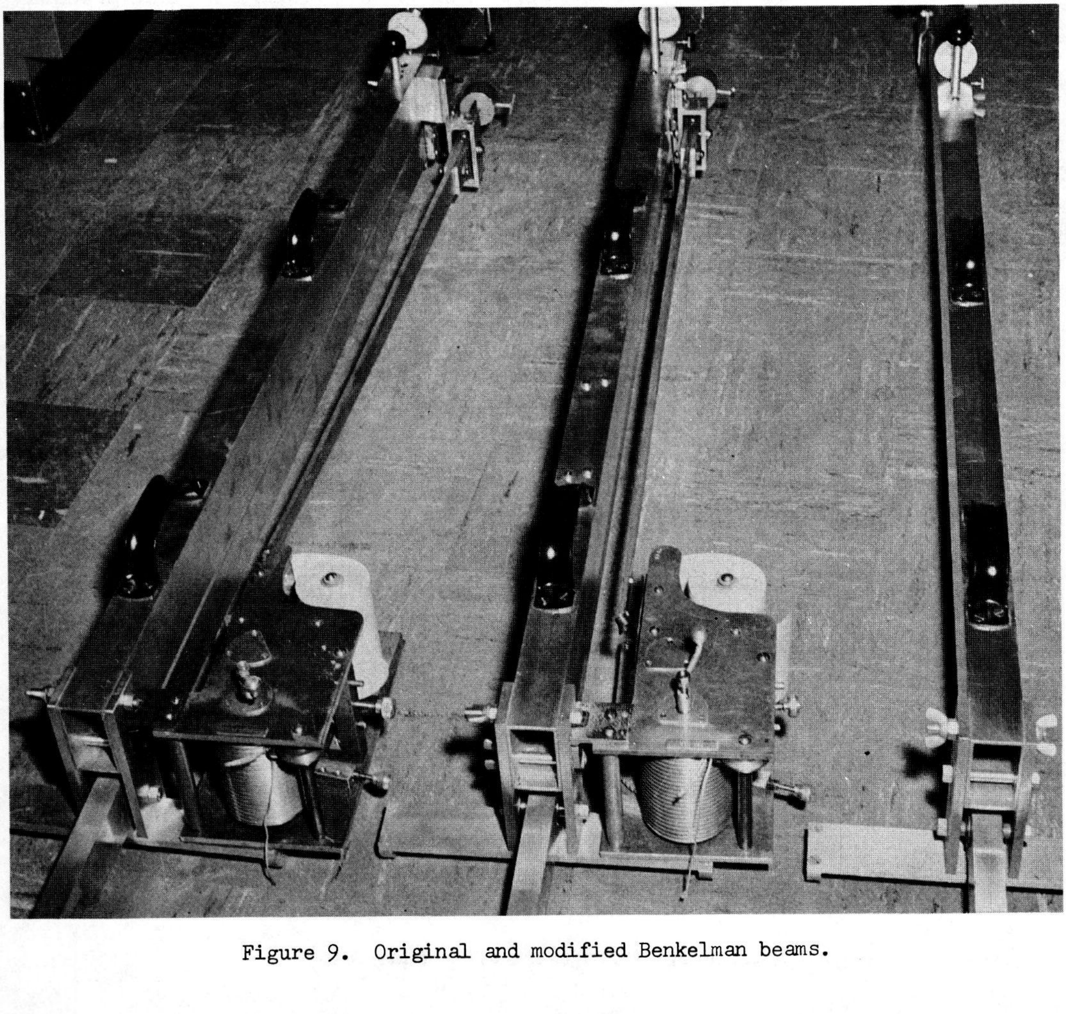

Figure 9. Original and modified Benkelman beams.

test in order to vibrate the instrument and minimize the possible adverse effect of friction in the bearings and elsewhere in the system.

A graphic recording attachment designed by R. A. Helmer of the Oklahoma Department of Highways was purchased from the Industrial Machine Company of Oklahoma City, Oklahoma. This device graphically records the pavement deflection curve, thus showing both the amount and rate of pavement deflection and recovery. The unit consists of a 42-in. long recording lever arm mounted at the fulcrum on ball bearings to give a 20 to 1 multiplication at the end of the arm holding the pen. The other end of the arm is connected by a linkage to the probe arm of the Benkelman beam at a point directly opposite the dial micrometer. The resulting ratio between the front end of the probe and the pen is 1 to 10. A string wound on a drum, one end of which is attached to the test truck, pulls the paper through the recorder as the truck moves ahead. The pen traces a line showing the deflection data on a 3 7/8-in. wide roll of calculator paper moving past the pen at the rate of 1.15 in. of paper movement for each foot of movement of the test truck. The recorded trace thus shows as a horizontal line having a dip in the middle.

The Nice bearings, No. 5368, originally supplied with the Benkelman beam were open and loose. New Departure bearings, No. 99500, a close-tolerance sealed bearing, were substituted, and new shafts were made to fit the bearings.

The linkage between the recorder arm and the probe arm, as supplied with the Helmer recorder, consisted of a bolt on the recorder arm which rests on a plate attached to the probe as shown on the left beam in Figure 9. An adjustable weight on the recorder arm keeps the rear of the arm heavy and the bolt in contact with the plate. It was found that this arrangement was not entirely satisfactory. The linkage was not sufficiently direct and adjustment of the pen on the paper was too slow.

A new linkage was designed and is shown on the middle beam in Figure 9 and in Figures 10, 11, and 12. This linkage consisted of a slotted aluminum arm mounted on the recorder arm with a New Departure No. 77036 ball bearing. A second aluminum arm with the same type of bearing was mounted solidly to the probe arm. The two aluminum linkage arms were connected by means of a bolt, special nut, washer, and wing nut in such a manner that a direct, easily and rapidly adjustable linkage was provided.

This linkage appeared promising during lab calibration. The beam was then taken to the field and a number of projects tested. Discrepencies were noted between maximum deflection values for a given load at the same point but with different beams, and it was obvious that more developmental work would be necessary. Comparisons were made with a Bureau of Public Roads beam and inconsistencies were found between the two. Some of the difference was felt to be caused by the drag of the recorder pen on the paper, inasmuch as the Bureau beam had no recorder. Tests were then run with

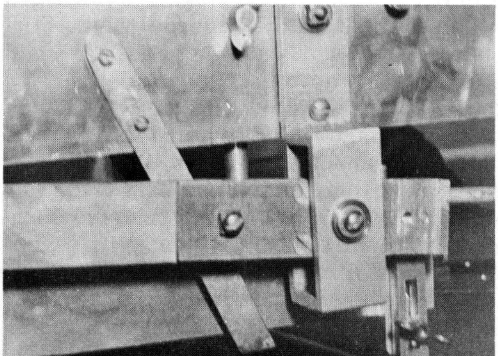

Figure 10. Unsatisfactory recorder arm linkage.

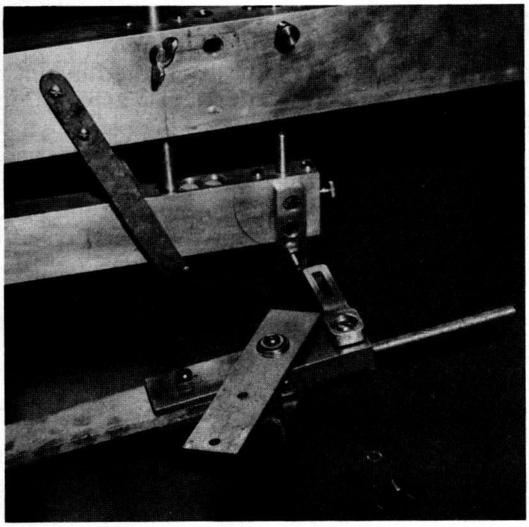

Figure 11. Unsatisfactory recorder arm linkage.

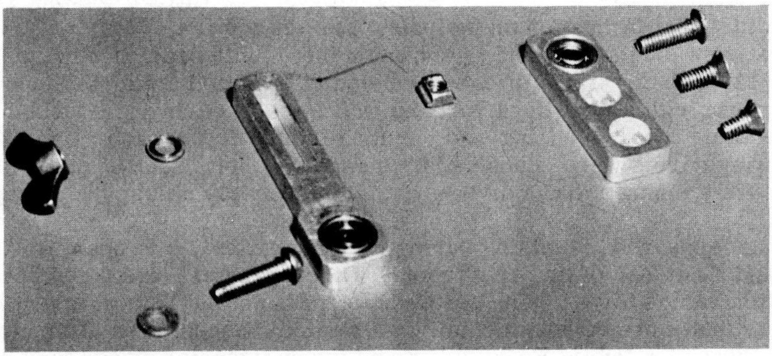

Figure 12. Unsatisfactory recorder arm linkage.

the recorder completely disconnected, with the recorder connected without using a pen, and with all components in operation. The linkage bearings proved to be the main problem, together with some drag from the recorder pen. Slight lateral displacement of the probe tip on the pavement caused binding of the bearings because no allowance was made in the linkage for lateral movement.

It was then decided to try a wire hookup, with the wire always in tension. Figures 13 and 14 show details of this linkage which is being used with satisfactory results at the present time. The wire is connected solidly to the recorder arm and extends down to the probe with an arrangement for rapid adjustment of the length of the wire to set the desired height of the pen. Experimentation in the size of the wire showed that, though not being critical, best results were obtained with piano wire approximately 0.032 in. in diameter and about 1.5 in. long. A somewhat surprising discovery was that the most accurate readings were obtained with the recorder arm almost in balance about the fulcrum. Apparently the wire is stiff enough to resist bending under small compressive loads, but will flex sufficiently to allow necessary lateral movement of the probe arm.

The South Dakota beams with the newly designed linkage showed excellent precision in the office. On returning to the field, comparison tests showed 84 percent of the

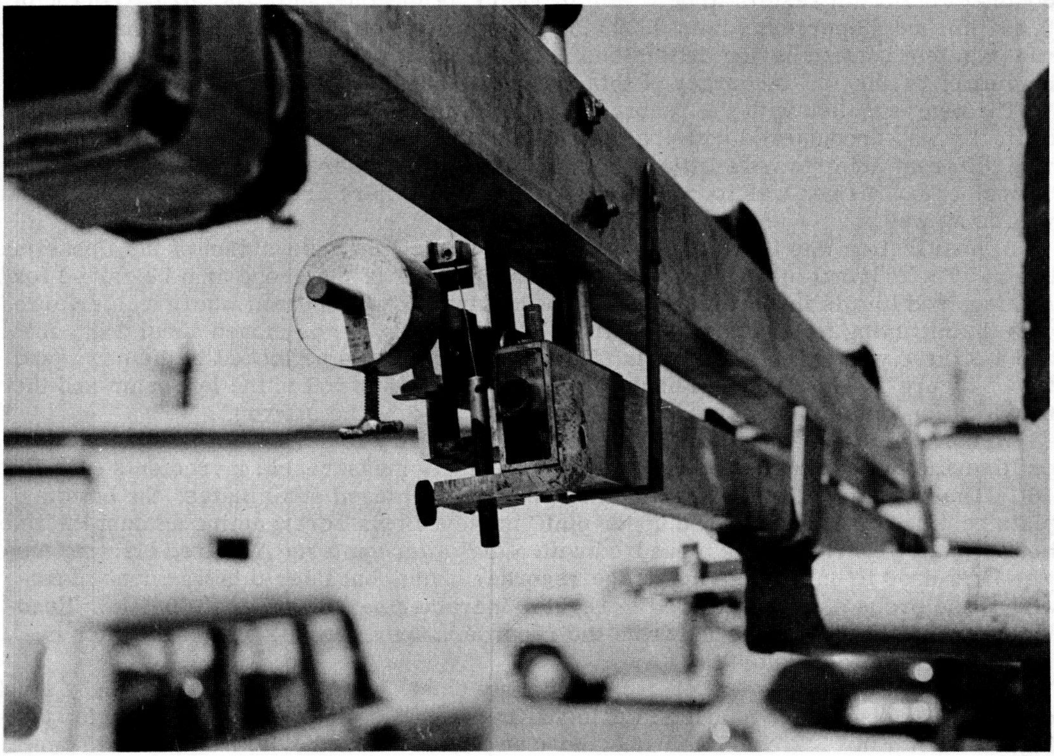

Figure 13. Satisfactory recorder arm linkage.

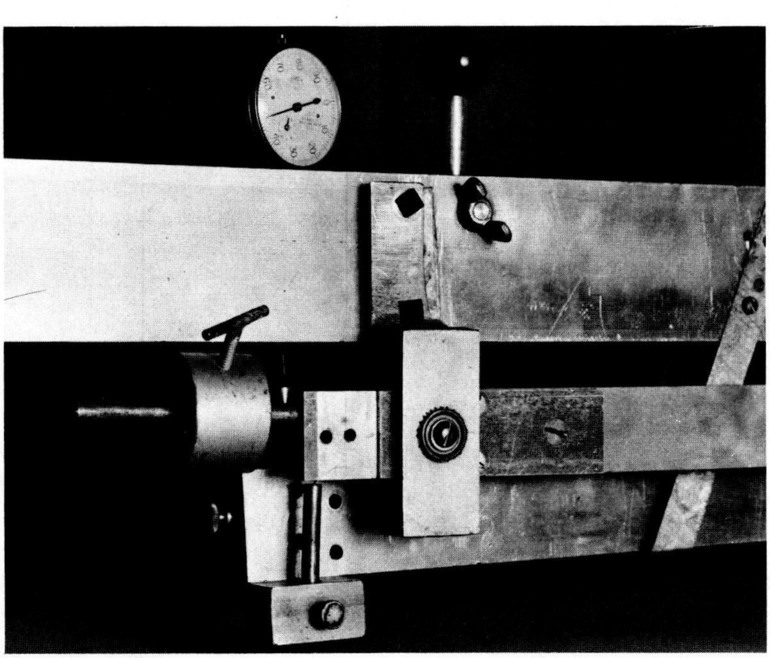

Figure 14. Satisfactory recorder arm linkage.

South Dakota beam results to be within 0.006 in. of the Bureau beam, 61 percent within 0.004 in. and 48 percent within 0.002 in. out of a total of 56 tests. All of the tests were run on a test section having deflections in the range of 0.040 to 0.080 in. for an axle loading of 24,000 lb. Accuracy of this order was considered quite satisfactory.

The pens supplied with the recorders were Esterbrook fountain pens. Considerable difficulty was encountered with the operation of the pens in the field. Experimentation with different points was of little help, so ballpoint pens were finally tried. A Venus ballpoint No. 101 pen was found satisfactory although there are undoubtedly others that will do as well.

A modification was found to be necessary in the locking mechanism of the Benkelman beam. The original mechanism required that the rear of the probe arm be raised for locking, and this is inconvenient. Also, the locking rod was located where it interferred with the mounting for the recording arm. Consequently it was moved about 2 in. ahead and a stirrup was added so the rear of the probe arm could be locked in the down position. Figure 15 shows the original position of the locking rod in the left beam and the new location in the right beam. Figures 10 and 11 show the stirrup.

To calibrate the beam in the office, it was necessary to devise a method of raising or lowering the end of the probe by small, precisely measureable increments (see Fig. 16). The probe point was placed on the upper of two hinged steel plates, the bottom plate resting on the floor. The upper plate is moved by a screw acting against the fixed bottom plate. Movement is measured with a dial micrometer. A geared electric motor was first used to pull the string on the recorder drum, but later this was done manually. The probe point was moved in regular increments of 0.005 or 0.0001 in. Readings were taken on the dial and from the recorder paper. In this way calibration between the two was possible (3).

The field crew consists of four men, a party chief, two beam operators, and a truck driver. The beams are carried in external racks on the sides of the truck during testing, as shown in Figures 17 and 18. Between test projects they are carried in compartments on top of the body as shown in Figure 17.

The truck used for testing is a two-axle Ford F-950 with a specially built compartmented tank body which can be loaded with water by a self-contained pump. The truck can be loaded to either side or end, with a maximum rear axle load of about 26,000 lb. The trucks are interchangeable between the plate load testing and Benkelman beam testing.

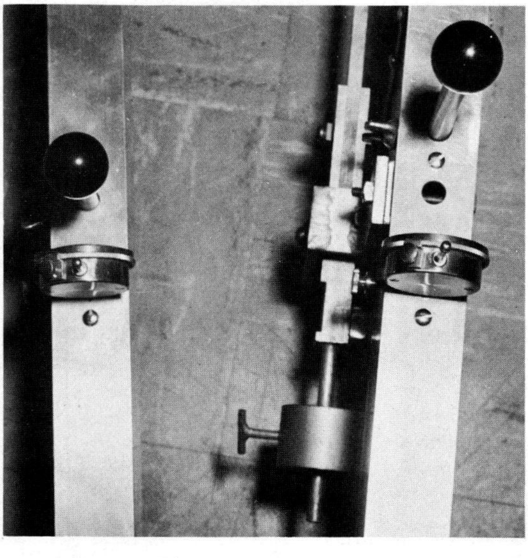

Figure 15. Locking rod location.

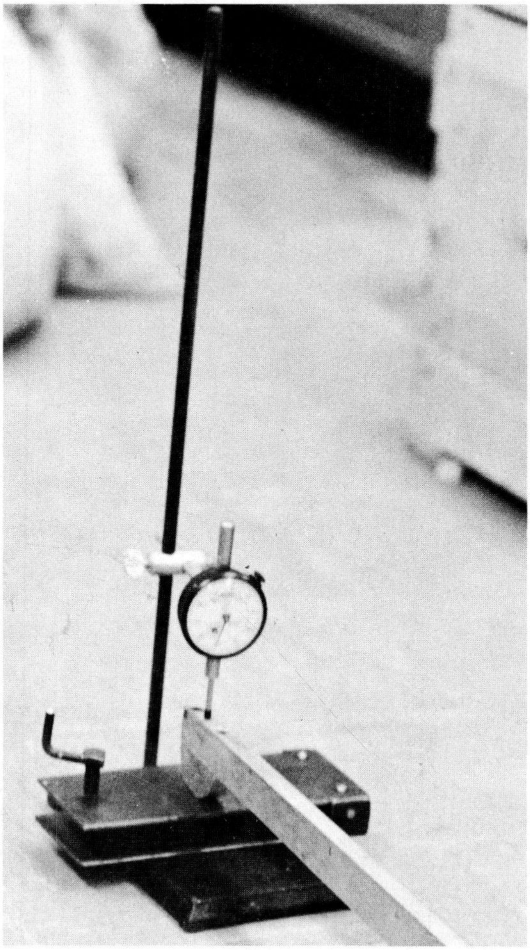

Figure 16. Beam calibration device.

The test sites are chosen to give a representative sampling of all conditions present in the road to be tested. There are 1033 test sites included, or 2 or 3 test sites per mile. There are from 12 to 50 test locations on a project, depending on its length. Grades of over 3 percent and superelevated curves are excluded because of the possible effect of weight transfer in the test trunk. In the interests of safety, sight distance is also considered. Each test site is marked on the outside wheel lane by a large T (see Fig. 19) so that tests can be repeated at nearly identical points. The outside dual of the truck is placed along the top of the T. For the initial test position, the truck is stopped with its rear axle opposite the stem of the T, as shown in Figure 20.

The "T's" are painted on the pavement in such a manner that the beam probe is located in the wheel paths most commonly used by traffic. These wheel paths are usually clearly visible due to darkening of the surface caused by traffic. The outer wheel path is usually 2 to 3 feet from the pavement edge on a 24 foot wide mat. The wheel paths on South Dakota Highways are closer to the center line than in some other states. This is probably due to the light traffic volume carried by South Dakota Highways. As a result the beam probe in the outer wheel path is usually located approximately 2 to 3 feet in from the pavement edge.

On stopping the truck at a test site, the beams are lifted off the racks and the probe inserted between the dual tires of each rear wheel. The tires are 12 x 24. 5, tubeless, 14 ply. The testing is done with 85-psi tire pressure and a 24, 000-lb axle load. The

Figure 17. Beam carrying rack.

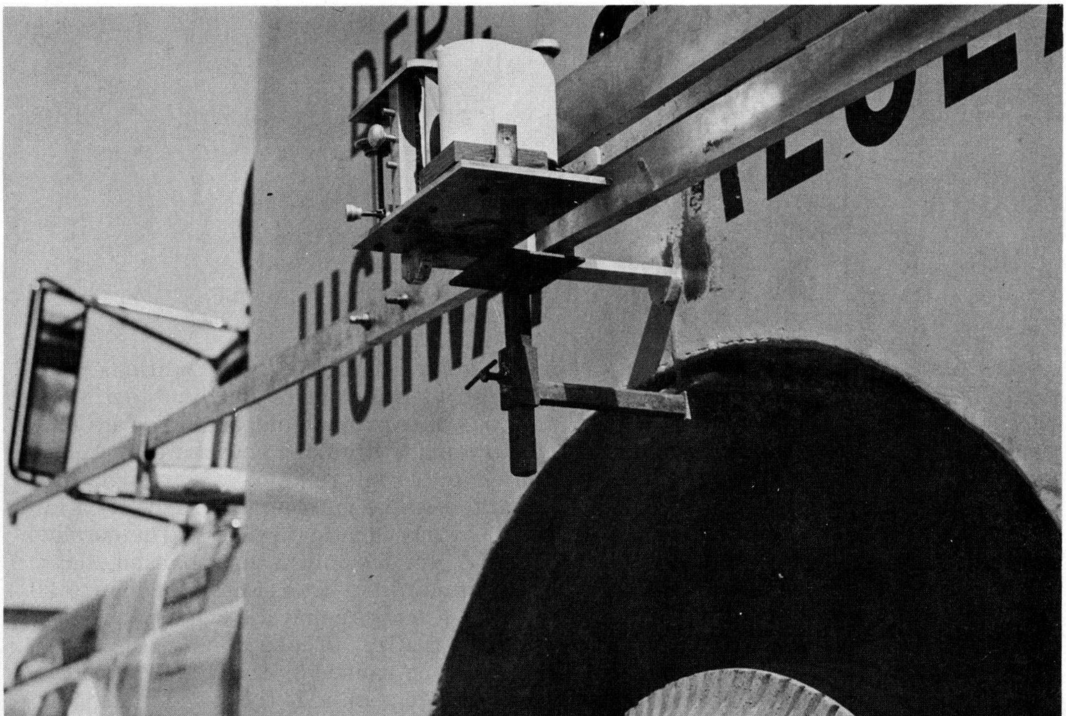

Figure 18. Beam carrying rack.

Figure 19. Beam test site.

probe point is positioned approximately 4.5 ft ahead of the center of the axle at the beginning of the test. The string is attached to the truck by a spring clip and pulls loose when the string is entirely unwound. After releasing the locking mechanism of the probe, the recorder arm is adjusted until the pen is within about 3/4 in. of the top of the paper. The pen is moved vertically to make a mark on the paper for the beginning of the test, the Ames micrometer dial is read, and the readings recorded by the party chief. The truck is then driven ahead at creep speed (about 2 or 3 mph) until the clips pull loose. Meanwhile, the beam operators note the dial readings at maximum deflection and again at the end of the test.

The pen is moved vertically by hand to show the end of the test. The record is then removed and given to the party chief in order that the project number, test site, date and wheel path may be noted upon it. The recorder sheets are processed in the office with a template which gives directly the value of the maximum deflection.

In 1961 two trucks were used in conducting the summer series of Benkelman beam tests. One truck had a wheel load of 7,500 lb and the other a wheel load of 12,000 lb. During this test series, 604 locations were tested. The 7,500-lb wheel load was used first and immediately behind it the 12,000-lb wheel load was used. The same beams were used for both loads. Table 3 contains some of the data obtained from this series of tests. If the deflection-wheel load relationship were linear it would be necessary to multiply the deflection readings obtained at a 7,500-lb wheel load by 1.600 to obtain the deflection caused by the 12,000-lb wheel load. Similarily it would be necessary to multiply the deflection readings taken at a 12,000-lb wheel load by 0.625 to obtain the 7,500-lb wheel load deflections. Actually, the relationship is not quite linear and correction factors for the 12,000- and 7,500-lb wheel loads for the average of these 604

Figure 20. Beam test site.

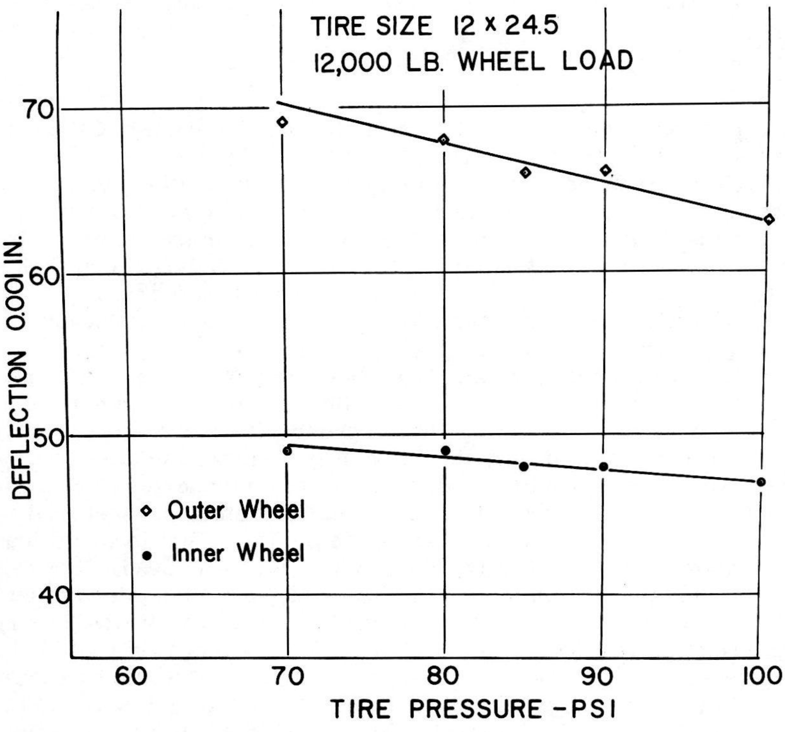

Figure 21. Tire pressure vs deflection.

TABLE 3
COMPARISONS OF WHEEL LOAD VS DEFLECTION

Project	Test Location (no)	Deflection (in x 10^{-3}) 12,000-Lb Wheel Load Actual IW	Actual OW	Estimated IW	Estimated OW	7,500-Lb Wheel Load Actual IW	Actual OW	Estimated IW	Estimated OW
2	35	49	81	49	76	29	45	29	48
4	35	23	32	24	34	14	20	13	19
5	19	67	92	61	90	36	53	40	54
6	20	52	68	52	67	31	40	31	40
7	38	46	58	47	56	28	33	27	34
8	25	50	58	47	59	28	35	30	34
9	13	31	42	32	42	19	25	18	25
10	37	33	43	33	46	20	27	19	26
12	19	44	51	44	51	26	30	26	30
14	32	52	65	53	72	31	43	31	39
16	30	37	51	36	54	21	32	22	30
17	31	42	50	41	48	24	29	24	29
18	32	37	42	35	38	21	22	22	25
19	26	70	79	65	88	38	52	41	47
20	27	34	47	33	48	19	29	20	28
21	30	56	70	53	75	31	44	33	41
23	12	18	28	25	34	15	20	11	16
24	28	58	67	54	66	32	39	34	39
27	29	99	113	104	124	61	73	58	67
28	23	43	49	42	48	25	29	26	29
29	22	62	89	59	91	35	54	37	53
30	41	30	37	29	40	17	24	18	22

TABLE 4
PAVEMENT DEFLECTIONS AT VARIOUS TIRE PRESSURES AND TEMPERATURES,
1 1961 TEST SERIES, INNER WHEEL, 12,000-LB WHEEL LOAD

Run No	Run Date	Tire Pressure (psi)	Pvt Temp. (°F)	Beam No	Pavement Deflection (0 001 in.) Test 1	Test 2	Test 3	Test 4	Test 5	Test 6	Test 7	Test 8	Test 9	Test 10	Avg
4	4-7	85	70	776	34	49	37	46	55	43	59	46	54	47	47
5	4-7	85	70	777	34	53	37	50	63	41	63	44	50	51	49
10	4-18	100	66	776	36	51	36	49	63	42	55	44	42	43	47
11	4-18	90	82	776	34	56	37	38	72	44	60	49	48	45	48
12	4-18	80	84	776	39	54	37	50	65	44	58	42	48	48	49
13	4-18	70	86	776	37	60	36	42	72	43	59	47	48	48	49
14	4-21	85	54	776	35	45	33	41	58	37	51	36	43	44	42
24	6-5	85	110	776	36	62	41	50	63	39	61	47	55	48	50
25	6-6	85	96	776	39	60	40	49	66	48	60	48	53	49	51
26	6-6	85	102	776	43	63	43	50	70	47	68	52	53	50	54
27	6-16	85	78	776	38	52	40	46	60	43	56	43	44	51	47

TABLE 5
PAVEMENT DEFLECTIONS AT VARIOUS TIRE PRESSURES AND TEMPERATURES, 1961 TEST SERIES, OUTER WHEEL, 12,000-LB WHEEL LOAD

Run No	Run Date	Tire Pressure (psi)	Pvt Temp. (°F)	Beam No	Pavement Deflection (0 001 in.) Test 1	Test 2	Test 3	Test 4	Test 5	Test 6	Test 7	Test 8	Test 9	Test 10	Avg.
4	4-7	85	70	779	52	81	79	60	74	60	72	56	50	50	63
5	4-7	85	70	775	50	79	81	67	76	64	66	54	50	50	64
10	4-18	100	66	779	52	78	75	62	75	63	66	45	55	54	63
11	4-18	90	82	779	58	70	85	58	83	62	68	60	64	55	66
12	4-18	80	84	779	60	81	95	65	76	67	76	56	53	50	68
13	4-18	70	86	779	56	84	92	67	78	67	74	52	58	59	69
14	4-21	85	54	779	50	67	72	61	77	58	65	50	51	52	60
24	6-5	85	110	779	71	87	90	64	77	66	70	61	58	55	70
25	6-6	85	96	779	58	84	92	67	84	68	72	58	58	50	70
26	6-6	85	102	779	62	89	99	69	86	70	78	60	60	52	73
27	6-16	85	78	779	55	73	80	61	70	57	66	53	50	50	61

tests are 0.591 and 1.690 respectively. Table 3 shows the actual deflections for loads at 7,500 and 12,000 lb and also the estimated deflections for each wheel load obtained by using the previously mentioned correction factors. The actual deflections are an average of the deflection readings taken at the number of test locations shown in the table. These data show that the deflection values obtained at a given wheel load can be corrected with reasonable accuracy to find the deflection caused by other wheel loads.

Besides determining the influence of wheel load on deflection, an attempt was made to find the effect of tire pressure and pavement temperature on deflection. Tire pressures were varied from 70 to 100 psi, the wheel load was maintained at 12,000 lb, and 10 locations were tested. The particular highway used for this series of tests has a class F surface 2 in. thick, 5 in. of base course, and 9 in. of subbase. The data obtained are given in Tables 4 and 5. The 85-psi tire pressure was run 7 times at different temperatures. The average deflection of these 7 runs is 0.066 in. This value was used together with the data for the other 4 runs given in Tables 4 and 5 to make the plot shown in Figure 21. This figure shows that the tire pressure does not greatly influence the deflection. However, for the conditions of the tests, there is a tendency for the deflection to decrease with an increase in inflation pressure. The reduction in deflection with increased tire pressure may be due to a greater tendency for the asphaltic concrete to squeeze up between the dual tires at the higher tire pressures.

The 7 runs conducted at 85-psi tire pressure and 12,000-lb wheel load were made at temperatures varying from 54 to 110 F. These data are plotted in Figure 22 and show that there is an increase in deflection with an increase in temperature for this particular surface. If this relationship holds true on other pavements and for other temperatures the deflection values could be corrected to a standard temperature of perhaps 70 F.

MISCELLANEOUS TESTS

Before any field testing was started, an attempt was made to determine how wheel load and tire pressure affect the contact area between the tire and the pavement. One

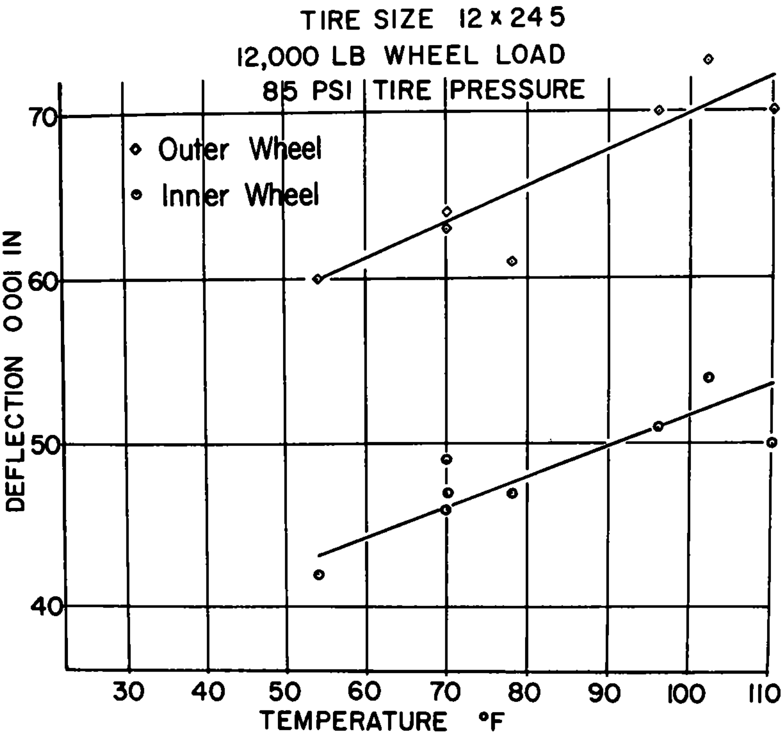

Figure 22. Temperature vs deflection.

wheel was jacked off the ground, the tire's tread was painted, and the painted tires lowered on cross-section paper. This procedure was repeated for tire pressures of 70, 80, 90, and 100 psi and for wheel loads of 6,000, 7,500, 9,000, 11,200, 12,000, and 13,000 lb. The total area of the prints of both tires was then measured. The data are given in Table 6 and are shown graphically in Figure 23. The data show that, as the wheel load increases, the contact area also increases, but apparently not in a linear manner. At a tire pressure of 80 psi the contact area is increased from 108 to 169 sq in. with an increase in wheel load from 6,000 to 13,000 lb. The data also show that as tire pressure is increased the contact area is decreased for any specific wheel load. At a 12,000-lb wheel load the contact area is decreased from 167 to 149 sq in. with an increase in tire pressure from 70 to 100 psi.

SUMMARY

This report was written to describe the flexible pavement research program being conducted in South Dakota. The study is intended to produce information for use in evaluating the current design method. Thirty-four representative sections of highway

TABLE 6

CONTACT AREA

Wheel Load	Contact Area (sq in) for Tire with											
	70-Psi Pressure			80-Psi Pressure			90-Psi Pressure			100-Psi Pressure		
	IW	OW	Both	IW	OW	Both	IW	OW	Both	IW	OW	Both
6,000	58	56	114	56	52	108	54	51	105	52	46	98
7,500	67	64	131	63	61	124	62	59	121	60	57	117
9,000	76	73	149	71	69	140	71	66	137	67	62	129
11,200	81	79	160	78	78	156	77	75	152	74	72	146
12,000	84	83	167	81	78	159	77	74	151	77	72	149
13,000	88	85	173	86	83	169	82	79	161	80	75	155

[1]Tire size 12·00 x 24.5 tubeless

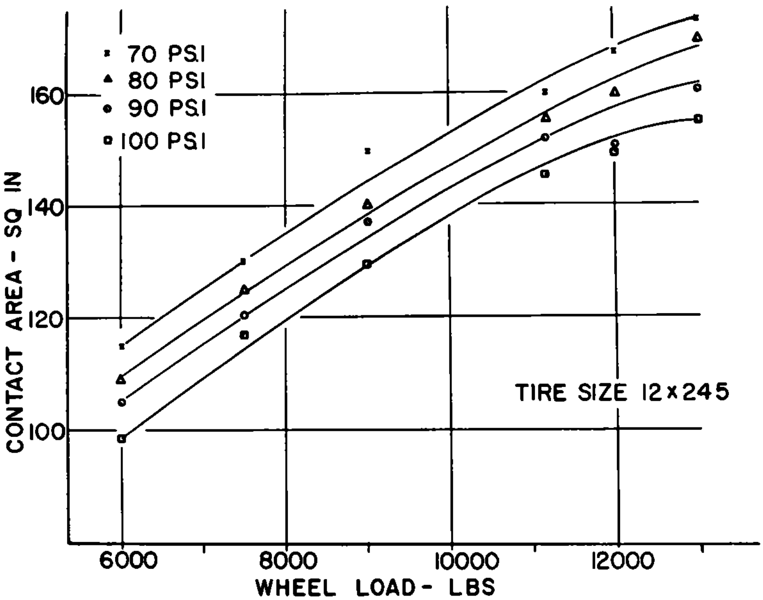

Figure 23. Tire contact area vs wheel load.

were selected for comprehensive investigation. The relationship of highway cost and performance to soil type, geology, type and frequency of traffic, climate, and construction material quality is being studied. Highway performance is being determined by condition surveys, road roughness determinations, plate load testing, and Benkelman beam tests. The general plan of the study and various test procedures being used are discussed. A few of the preliminary findings have been presented; however, due to the volume of the data being obtained no attempt has been made to present detailed information nor to enter into a comprehensive analysis of the data presented.

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Significance of Layer Deflection Measurements

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An understanding of pavement behavior is essential to the development of an effective method of pavement design. To this end, a system for evaluating the structural performance of existing pavements is required. One system of evaluation and its effectiveness is described in this paper.

Data obtained from a test road located on US 31 near Columbus, Indiana, were used to develop the evaluation methods. Procedures such as the analysis of existing crack patterns and wheel track rutting and their relationships to subgrade soil type were examined. Total surface deflections under load, measured with a Benkelman beam, were analyzed in an attempt to establish a relationship between deflection and cracking of the pavement surface.

Failure to establish total deflection as an indicator of the pavement behavior led to the development of a method using the Benkelman beam to measure deflections of the individual layers of the pavement structure. Four-inch holes were drilled to the interface of the different layers of the pavement, and the holes were cased with pipe. Steel rods were referenced at the bottom of each hole, extending upward to near the top of the pavement. Measurements were made under rear axle loads of 12,000, 18,000, 22,000 and 27,000 lb.

Relative modulus values using layer deflections were calculated to compare the relative deflection of one pavement layer with another. Theoretical stress distribution was used as a basis of the calculations.

The important conclusions reached by this study were that total deflections were ineffective in establishing the cause of the flexible pavement cracking and that knowledge of the individual layer deflections was required in order to evaluate the pavement fully.

● THE AMOUNT a flexible pavement deflects under load indicates, in part, its adequacy insofar as structural capacity is concerned. Repeated deflection may cause the pavement to crack and distort as a result of (a) fatigue, (b) excessive bending stresses, and (c) accumulated plastic deformation and other factors.

The deflection of a flexible pavement is partly elastic in character, but it is also made up of plastic strains. Elastic strains are regained on removal of an applied load whereas plastic strains are not. Thus, the accumulation of these nonrecoverable plastic strains with repeated applications of load can result in distortion of the paving surface.

It must be recognized at the outset that performance of a flexible pavement is influenced by many factors. These include gross load, tire pressure, repetition of load, thickness and quality of the various pavement components, and the elastic plastic properties of the pavement components (particularly the subgrade soil). Pavement failure may result from excessive shear stresses, vertical deflection, or a combination of these.

Several methods of flexible pavement design are based on limiting deflection criteria.

These include procedures adopted by the Kansas State Highway Department and the Navy Department. Both of these methods of design are predicted in part on theoretical considerations that relate pavement stresses and deflections to the applied load. Certain simplifying assumptions are made regarding the shape of the tire imprint upon the pavement surface, the relationship between tire pressure and contact pressure and homogeneity and isotropy of the structural system.

Many engineers use deflection measurements to evaluate the adequacy of existing pavements. The literature contains numerous references to deflection measurements, including the work done on the WASHO and AASHO Road Tests. Deflection measurements are but one tool that can be used by the researcher to formulate concepts regarding the behavior of flexible and rigid pavements. Deflection measurements are subject to many limitations and therefore must be considered to be a means towards an end rather than an end within themselves.

The primary purpose of determining the deflection of an existing pavement, insofar as structural adequacy is concerned, is to obtain basic data, either by inference or direct measurement, relative to the stress-strain properties of the pavement materials. Mere measurement of gross deflection at the pavement surface may not yield the desired results. Such factors as pavement layer deflection, radius of bending and the viscoelastic properties of the pavement components must also be considered.

To be of maximum benefit to the engineer, deflection measurements must be planned so that a large amount of information is obtained without resorting to elaborate field installations. This is true inasmuch as the time required to install deflection gages in pavements is great, which in turn limits the number of measurements that can be obtained.

Surface deflection is made up of cumulative deflections of all the pavement components including the subgrade. Also, for the usual case a large portion of the deflection occurs in the subgrade. It is to be noted that the pavement may tend to "heave" both between and outside the dual wheels.

As depth increases, the profile of bending changes from that found immediately under the wheels and is saucer shaped. Surface deflection results from an accumulation of strains from the surface downward; the distance a particle moves when a load is applied at the surface decreases with depth.

It appears that measurement of surface deflection alone may be misleading in some cases unless depth of the layer contributing the largest portion of the deflection is known. As a rule, if the major portion of the deflection is in the subgrade, large radii of curvature occur, whereas small radii result if the deflection occurs in the upper layers of the pavement. Because tensile stress varies inversely with radii of curvature of the deflected surface it is apparent that knowledge of depth of the deflected layer is important.

A series of deflection measurements were made on the US 31 Test Road near Columbus, Indiana. The purpose of making these measurements was to determine the significance of layer deflection and in particular to ascertain whether correlations could be established between total deflection and pavement condition and between layer deflection and pavement condition. A large number of surface deflections were determined with the Benkelman beam. Layer deflections (that is, top of base, subbase and subgrade) were made at selected locations. Comparisons were made among surface rutting, cracking, surface deflection and layer deflections.

The test pavements offered an excellent opportunity to study these comparisons because the pavement was constructed under closely controlled construction conditions. An intensive testing program was carried out during the planning and construction phases. Detailed information was available regarding soil conditions, construction problems, strength properties of all pavement components and perhaps most important, detailed information on the pavements' performance was available.

DESIGN AND CONSTRUCTION OF TEST PAVEMENT, US 31

The flexible pavement design was based on a combination of the Corps of Engineers CBR and the Group Index methods. Subgrade types ranged from A-1-a sands and gravels to plastic A-6 clays (AASHO soil classification). The basic design of the pavement structure was as follows:

1 in. — asphaltic concrete surface course,
1½ in. — asphaltic concrete binder course,
2½ in. — asphaltic concrete base course,
8 in. — waterbound macadam base course, and
5 in. — (inside edge) to 8 in. (outside edge) — open-graded drained granular subbase course.

The subgrade, after compaction to 100 percent of the maximum standard AASHO value, received at least two coverages of a heavy pneumatic roller (20- to 35- tons gross load and 50- to 70- psi tire pressure).

The subbase, which was open-graded and had an average of 2.8 percent fines passing the No. 200 mesh sieve, was compacted with a multiple shoe vibrator to 100 percent of maximum standard AASHO density. Because the subbase lacked cohesion, the heavy pneumatic compactor could not be used. The multiple shoe vibrator was not employed until the top 2 1/2 in. were mixed with 70 lb per sq yd of limestone screenings.

Crushed stone was used in the construction of the waterbound macadam base course. Each layer was compacted with a multiple shoe vibrator and a heavy pneumatic roller. Two complete coverages were made with the heavy pneumatic roller loaded to a gross weight of 35 to 50 tons, producing contact pressures of approximately 70 to 85 psi.

A 60- to 70- penetration asphalt was used in all three bituminous concrete layers, 4.5, 5, and 6 percent for the base, binder, and surface courses, respectively. All pavement was placed during the 1953 construction season. The final section was opened to traffic December 11, 1953.

OBSERVANCE OF CRACKING AND RUTTING

Early in 1957, longitudinal cracking began to appear at several locations of the flexible pavement. The cracking generally occurred in both the outer and inner wheel tracks. Where the cracking was severe, some transverse cracking was apparent. Wheel track rutting was observed at the same time as the cracking. In general the rutting did not exceed 0.5 in.

CRACKING, RUTTING, AND SURFACE DEFLECTION STUDY

Initial studies of the test pavement included an analysis of the extent of cracking, rutting, and measurement of surface deflection.

Cracking and Soil Type

Table 1 gives the linear feet of cracks per station (100 ft) for the four basic soil types found under the flexible pavement. All types of cracks are included except those which seemed to delineate the pavement centerline giving the appearance of a plane of weakness or a "cold joint."

TABLE 1

RELATIONSHIP BETWEEN CRACKING AND SOIL TYPE, JANUARY-FEBRUARY 1960

Soil Type	No. of Stations	Linear Ft of Cracks	Avg. Linear Ft of Cracks per Station
A-1	39	4,814	123
A-2	53	5,863	111
A-4	177	23,901	135
A-6	109	11,138	102
All	378	45,716	121

Table 1 shows that more cracks occurred in the pavement built over the A-4 subgrade than pavement built on the other subgrade types. Also shown is that fewer cracks have resulted in pavement built on A-6 subgrades.

Other crack data not presented here have shown that more cracks have occurred in the traffic lane than in the passing lane, and that cracking is not directly related to pavement surface thickness. Also, observation has shown that the cracking has progressed since first appearing.

Surface Deflection Study

During the first week of June 1959, and the second week of May 1960, Benkelman beam deflection measurements were made at the test road site.

Deflection measurements were made at 48 carefully selected locations. Thirty-two of these locations fit into a pattern of variables given in Table 2. The variables selected were bituminous pavement surface thickness, crack frequency, lane, and soil type. In general, a pavement showing high crack frequency had 80 or more lineal feet of cracks per station. A section of pavement showing 25 or less lineal feet of cracking was classified as having low crack frequency. The nominal bituminous pavement thickness was 5 in. and this was used as the demarcation line for the surface thickness variable. The sites were also selected on the basis of soil type.

TABLE 2

DESIGN OF EXPERIMENT FOR BENKELMAN BEAM DEFLECTION MEASUREMENTS

Soil Type	Lane	Crack Frequency	Bituminous Pavement Thickness (in.)
A-1	Traffic	Low	5+
			5-
		High	5+
			5-
	Passing	Low	5+
			5-
		High	5+
			5-
A-2	Etc.		
A-4	Etc.		
A-6	Etc.		

Deflections were measured under an 18,000-lb rear axle load. Deflections reported in this paper are those taken between the tires of both the rear dual wheels of the truck. In taking the measurements, the truck, initially 10 ft away, was backed to the point of measurement so that its rear axle passed approximately 3 in. beyond the point, and then it was moved forward. In this manner, the rear wheels came no closer than 7.5 ft to the reference feet of the Benkelman beam. In only four out of several hundred tests did a dial reading of other than zero result when the truck was 7.0 or more feet away from the probe of the beam. It is therefore believed that significant movement of the reference feet did not occur.

Table 3 presents the combined 1959 and 1960 inner and outer wheel path data. A four-way classification analysis of variance was made to study the effects of soil type, surface thickness, crack frequency and lane type on the deflection data. Significance of a factor was determined by variance ratios or "F" tests at 0.05 and 0.01 levels. For

TABLE 3

SUMMARY OF TOTAL DEFLECTION FOR 1959 AND 1960 DATA, INNER AND OUTER WHEELPATH

Crack Frequency	Bituminous Pavement Thickness (in)	Total Deflection (10^{-3}in)										
		A-1 Soil		A-2 Soil		A-4 Soil		A-6 Soil		Total		
		Traffic Lane	Passing Lane	Traffic Lane	Passing Lane	Traffic Lane	Passing Lane	Traffic Lane	Passing Lane			
Low	5+	15	15	17	9	20	13	24	14	127		
		18	18	17	18	32	17	32	22	174		
		14	14	12	11	18	14	20	12	115	559	
		16	16	15	15	20	18	25	18	143		1, 200
	5-	21	13	20	16	30	24	22	17	163		
		20	13	19	19	30	36	21	21	179		
		20	11	16	10	25	19	18	16	135	641	
		20	14	19	20	24	26	19	22	164		
High	5+	11	12	17	11	26	10	18	13	118		
		10	14	26	12	26	16	19	30	153		
		13	15	15	13	20	10	14	17	117	533	
		12	12	22	18	23	22	19	17	145		1, 144
	5-	14	16	18	12	24	20	34	15	153		
		15	11	18	13	24	22	34	17	154		
		15	13	17	10	25	18	34	14	146	611	
		15	13	17	8	24	24	34	23	158		
Total		249	220	285	215	391	309	387	288	2, 344		
		469		500		700		675				

TABLE 4

FOUR-WAY ANALYSIS OF VARIANCE RESULTS FOR 1959 AND 1960 DEFLECTION DATA, INNER AND OUTER WHEELPATH

Source of Variance	F	$F_{0.05}$	$F_{0.01}$	Significance
Soil type, A	36. 40	2. 71	4. 03	0. 01 level
Lane, B	50. 98	3. 95	6. 96	0. 01 level
Crack frequency, C	2. 00	3. 95	6. 96	NS
Surface thickness, D	16. 66	3. 95	6. 96	0. 01 level
Interactions A x B	2. 33	2. 71	4 03	NS
A x C	2. 92	2. 71	4. 03	0. 05 level
B x C	1. 33	3. 95	6. 96	NS
A x B x C	2. 17	2. 71	4. 03	NS
A x D	3. 75	2. 71	4. 03	0. 05 level
B x D	1. 08	3. 95	6. 96	NS
C x D	0	3. 95	6. 96	NS
B x C x D	5. 00	3. 95	6. 96	0. 05 level
A x B x D	4. 83	2. 71	4. 03	0. 01 level
A x C x D	7. 00	2. 71	4. 03	0. 01 level
A x B x C x D	7. 91	2. 71	4. 03	0. 01 level

the effect of soil type on deflection, the Tukey method for determining a studentized range allowance for a set of means was applied (1).

Table 4 summarizes the results of the analysis of variance of the data presented in Table 3.

The statistical analysis shows that soil type, lane, and bituminous pavement thickness have an effect on the deflection data. Deflections on the traffic lane were higher than those on the passing lane and those on thick pavement surfaces were lower than those on thin pavement surfaces. Deflections were shown not to be related to crack frequency.

The Tukey analysis showed that real differences exist in deflection measurements made on course- and fine-grained soils, but that differences between the A-1 and A-2 were not significant or the differences between the A-4 and A-6 soils were not significant. Deflections on A-1 and A-2 subgrades were lower than those on A-4 and A-6 subgrades.

The interactions shown to be significant in the analysis of variance were mostly due to a fairly small error mean square. This is not unusual when a large number of values, such as in this case, are used in the analysis.

Wheel Track Rutting Study

During the first week of August 1959, a transverse profilometer constructed by the Bureau of Materials and Tests of the Indiana State Highway Department was used to obtain the wheel track rutting measurements at the same 48 locations used in the surface deflection study. A rutting value was determined for each location by determining the maximum difference in elevation for that location in inches.

Table 5 presents a summary of the wheel track rutting data. Table 6 summarizes the results of the analysis of variance on these data. As with the deflection data, the rutting data are affected by lane position and soil type but not by crack frequency. Higher rutting values were recorded in the traffic lane than in the passing lane. Unlike the deflection data, the rutting data were unaffected by pavement surface thickness.

Tukey's method was applied in the same manner as in the deflection data in order to determine which soil types might be significantly affecting the rutting data. Real differences exist in rutting measurements made on A-1 subgrades and the fine-grained subgrades. The A-2 soil group was so variable that it could not be established as performing differently from any of the three other soil groups. Thus, rutting measurements of pavement built on A-1 subgrades are significantly lower than those of pavements on A-4 and A-6 subgrades.

TABLE 5

SUMMARY OF INNER AND OUTER WHEEL TRACK RUTTING DATA

Crack Frequency	Bituminous Pavement Thickness (in.)	Rutting (10^{-2} in.) A-1 Soil Traffic Lane	A-1 Soil Passing Lane	A-2 Soil Traffic Lane	A-2 Soil Passing Lane	A-4 Soil Traffic Lane	A-4 Soil Passing Lane	A-6 Soil Traffic Lane	A-6 Soil Passing Lane	Total		
Low	5+	13	11	27	22	36	33	62	25	229	439	
		33	16	28	22	21	31	33	26	210		851
	5-	40	11	44	12	44	24	53	18	246	412	
		27	10	26	17	30	16	24	16	166		
High	5+	16	11	34	21	62	23	60	25	252	449	
		12	12	33	23	38	24	31	24	197		945
	5-	28	19	41	31	61	22	79	21	302	496	
		15	14	22	26	35	24	40	18	194		
Total		184	104	255	174	327	197	382	173	1,796		
		288		429		524		555				

TABLE 6

FOUR-WAY ANALYSIS OF VARIANCE RESULTS FOR INNER AND OUTER WHEEL TRACK RUTTING DATA

Source of Variance	F	$F_{0.05}$	$F_{0.01}$	Significance
Soil type, A	7.83	2.92	4.51	0.01 level
Lane, B	34.00	4.17	7.56	0.01 level
Crack frequency, C	1.20	4.17	7.56	NS
Surface thickness, D	0.06	4.17	7.56	NS
Interactions: A x B	2.00	2.92	4.51	NS
A x C	0.84	2.92	4.51	NS
B x C	0.19	4.17	7.56	NS
A x B x C	1.75	2.92	4.51	NS
A x D	0.38	2.92	4.51	NS
B x D	1.95	4.17	7.56	NS
C x D	0.74	4.17	7.56	NS
B x C x D	0.76	4.17	7.56	NS
A x B x D	0.08	2.92	4.51	NS
A x C x D	0.30	2.92	4.51	NS
A x B x C x D	0.86	2.92	4.51	NS

Relationship Between Deflection and Rutting

As can be seen by Figure 1, there is a good relationship between maximum deflection and wheel track rutting. The fitted line of Figure 1 was obtained by the method of least squares. It should be noted, however, that the points in this figure represent average values, and that the data for the A-4 soil are erratic.

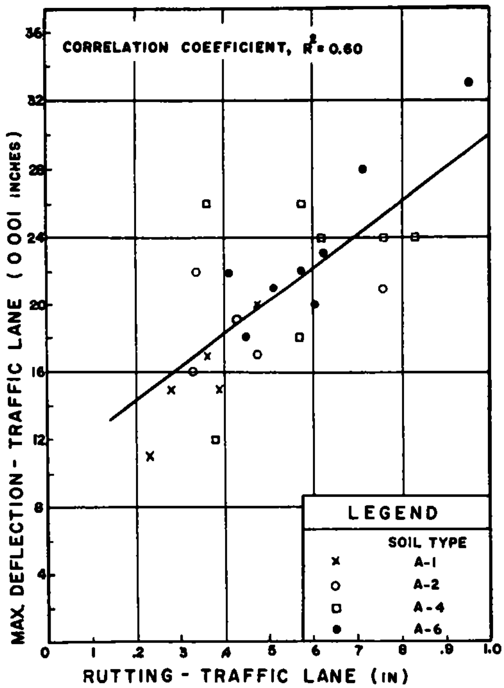

Figure 1. Maximum deflection vs rutting.

Summary

Concerning the cracking study, the following summary statements are applicable:

1. Cracking was not shown to be related to subgrade type.
2. More cracking occurred in the traffic lane than in the passing lane, no doubt due to heavier, and greater traffic volumes on the traffic lane.
3. Bituminous pavement thickness was not shown to be related to cracking.

The following statements summarize the surface deflection and rutting results:

1. Subgrade type, lane, and pavement thickness affected the deflection data.
2. Subgrade type and lane affected the rutting data.
3. Crack frequency was not shown to be related to total deflection or to rutting.
4. A direct relationship existed between rutting and total deflection.

LAYERED SYSTEM DEFLECTION STUDY

It is significant to note the previous study indicated that no correlation existed between cracking and surface deflection. Further, a correlation did exist between rutting and deflection and between rutting and soil type. The data suggest that surface deflection was directly related to soil type, fine-grained subgrades resulting in the highest deflection. Also rutting was influenced by soil type in the same manner. Thus it is indicated that the major portion of the rutting occurred in the subgrade itself. However, the probable cause of the longitudinal cracking was not apparent from the study.

The next step in the investigation concerned the study of deflection patterns within the component layers of the pavement. It was hypothesized that if the deflection of each pavement layer was determined at all of the sites previously tested, a relationship between these deflection data and pavement cracking could be established. Through the use of the stress distribution theory, relative moduli values, E, could be derived from the deflection data. The term relative modulus is used instead of elastic modulus, inasmuch as the pavement structure is imperfectly elastic. It should be noted that it was necessary to assume that materials under the cracked pavement underwent the same relative changes, with time, as the materials under the pavement that did not crack.

A scheme was devised in which the Benkelman beam was adapted to measure the vertical movement of steel rods referenced at the top of the different layers of the pavement system. This scheme (Fig. 2), used 4-in. diameter holes, with pipe casing and steel reference rods of varying lengths.

It was considered impractical to install a complete set of reference rods at all 32 sites of the previously discussed experiment; therefore, only eight sites were selected. The tests were made at eight locations in the passing lane where the surface was ap-

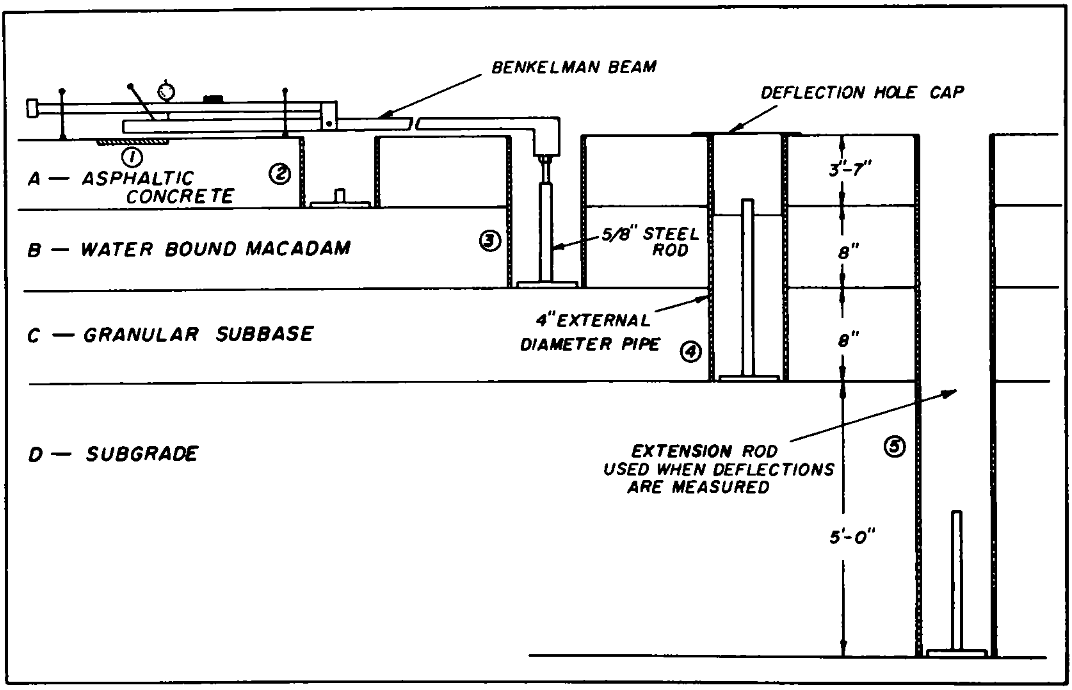

Figure 2. Use of Benkelman beam for layered system deflection study.

proximately 5 in. thick. Because of the importance of soil type, and because crack frequency was the factor for which an existing relationship was sought, these two factors were retained in the experiment. The eight sites selected for this study conform to the pattern of variables given in Table 7. All of the field work was completed during the months of September 1960 and April 1961.

TABLE 7

DESIGN OF EXPERIMENT FOR LAYERED SYSTEM DEFLECTION STUDY*

Soil Type	Station No.	
	Low Crack Frequency	High Crack Frequency
A-1	438, F-1	426, F-1
A-2	453, F-1	460, F-1
A-4	275, F-1	304, F-1
A-6	349, F-1	347, F-3

*All measurements were made in the inner wheelpath of the passing lane where the bituminous pavement was approximately 5 in. thick.

Test Procedures

Four different truck loadings were used: (1) 6,000 lb, right rear dual wheels, September 1960; (2) 11,250 lb, right rear dual wheels, September 1960; (3) 13,390 lb, right rear dual wheels, September 1960; and (4) 12,000 lb, right rear dual wheels, April 1961. The testing procedures were essentially the same as used for the surface deflection study; that is, the truck was backed from a distance 10 ft from the deflection point to a position over the point and then forward again.

Relative Moduli Values

Relative moduli values were obtained for the subgrade, subbase, and base courses for six of the eight locations. These are relative values only, because they are based on Boussinesq stress distribution and on the deflection measurements made through holes cased with pipe. Relative moduli offer a means of comparing the stress-strain properties of one layer to another.

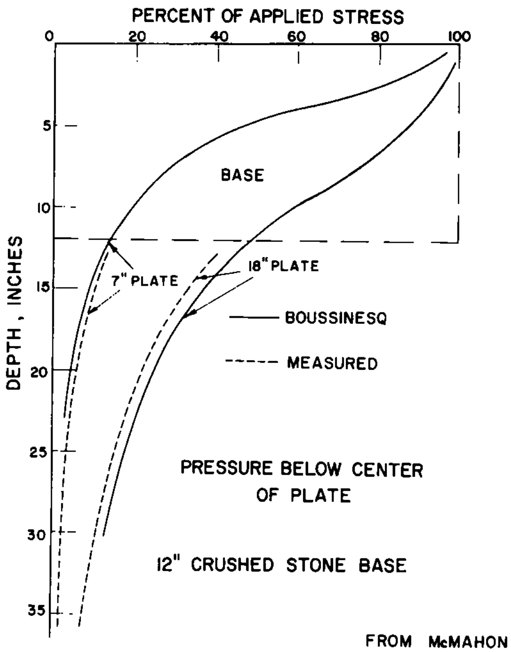

Figure 3.

It is recognized that in calculating these relative moduli values, application of elastic theory is made to a problem involving the stress-displacement properties of an imperfectly elastic material. However, inasmuch as the deflections dealt with in this study are not of the permanent type, but are primarily recoverable, the application of elastic theory to this problem appears justified.

Additional justification for use of the theoretical stress distribution computation to this problem is based on the fact that although numerical values of stress vary from the calculated values, the measured stress-depth curves are shaped similar to calculated curves (2, 3). Also changes in stress with depth are used in the moduli computations. Thus, because the measured and calculated curves have the same shape, the amount of error which is introduced into the computation is minimized. Figure 3 illustrates the comparison of curves obtained by test to calculated curves.

Considering the magnitude of loads received by the average highway, the use of elastic theory in the analysis of pavement

TABLE 8

SUBGRADE MODULUS, E_3, IN 1,000 PSI

Station	Dual Wheel Load, (lb)			Crack Frequency	Soil Type
	6,000	11,250	13,390		
347	32.8	20.5	19.1	High	A-6
249	26.4	27.5	19.0	Low	A-6
304	26.4	27.4	23.8	High	A-4
275	43.4	30.7	31.9	Low	A-4
460	32.8	32.5	31.9	High	A-2
453	21.7	27.4	31.9	Low	A-2
426	21.7	40.6	35.8	High	A-1
438	26.4	40.6	35.8	Low	A-1

TABLE 9

RESULTS OF TWO-WAY ANALYSIS OF VARIANCE OF SUBGRADE MODULI VALUES

Source of Variance	F	$F_{0.1}$	$F_{0.25}$	Significance
Soil type, A	2.37	2.49	1.50	0.25 level
Crack frequency, B	0.33	3.07	1.42	NS
Interaction: A x B	1.48	2.49	1.50	0.25 level

TABLE 10

SUBBASE MODULUS, E_2, IN 1,000 PSI

Station*	Dual Wheel Load (lb)			Crack Frequency	Soil Type
	6,000	11,250	13,390		
249	13.4	12.8	26.8	Low	A-6
304	13.4	12.8	13.2	High	A-4
275	10.8	14.9	13.4	Low	A-4
460	17.7	13.9	17.7	High	A-2
426	10.8	12.8	13.4	High	A-1
438	26.1	30.8	35.4	Low	A-1

*Station 347 and 453 had to be eliminated from this analysis because of defective hole installation referenced on top of subbase.

TABLE 11

RESULTS OF TWO-WAY ANALYSIS OF VARIANCE OF SUBBASE MODULI VALUES

Source of Variance	F	$F_{0.1}$	$F_{0.25}$	Significance
Wheel load, C	0.70	2.81	1.53	NS
Crack frequency, B	3.76	3.18	1.46	0.1 level
Interaction, B x C	0.43	2.81	1.53	NS

structures can be quite valuable, especially after the highway has been subjected to several years of traffic. During the past few years, several research efforts have produced results which point toward the concept that under any single application of a moving wheel load on a pavement surface, nearly elastic behavior exists (4). Deformation of the subgrades on the WASHO Road Test was elastic-like in that practically equal and recoverable deflections were produced by several thousand loads following the conditioning period by initial loads (5).

The equation used to compute moduli values was as follows:

$$S = \frac{pa}{E} F \tag{1}$$

in which

S = vertical elastic deflection, in.;

E = modulus of elasticity of the material (herein called relative modulus), psi;

p = unit load over circular area, psi;

a = radius of circular area, in.; and

F = deflection factor dependent on depth and radius of contact.

Values of deflection factor, F, can be determined for any depth or offset value using influence charts developed by Foster and Ahlvin (6) from work by Newmark (7).

Contact Pressures. — For the purpose of calculating the elastic moduli values, the contact pressure, p, was assumed to be equal to the tire pressure. The contact area between the wheel and pavement was assumed to be circular.

Test Results

The moduli for each layer that were calculated from the September 1960 data were analyzed by a two-day analysis of variance. Significance of a factor was determined by variance ratios or "F" tests (1). Tables 8 through 13 summarize the moduli results and the statistical analysis. Inasmuch as the April 1961 data were limited to one wheel load, the resulting moduli data are given in Table 14 for comparison purposes only.

The moduli of the bituminous courses were not determined from the deflection measurements because of the insensitivity of the Benkelman beam. In many cases no deflection was measured within the bituminous pavement; this resulted in calculated moduli equal to infinity.

TABLE 12

BASE COURSE MODULUS, E_1, IN 1,000 PSI

Station*	Dual Wheel Load, (lb)			Crack Frequency	Soil Type
	6,000	11,250	13,390		
249	49.7	101.2	132.0	Low	A-6
304	49.7	101.0	73.2	High	A-4
275	55.2	50.6	28.1	Low	A-4
460	27.6	65.0	47.2	High	A-2
426	55.2	50.6	69.4	High	A-1
438	17.8	24.0	47.2	Low	A-1

*Station 347 and 453 had to be eliminated from this analysis because of defective hole installation referenced on top of subbase.

TABLE 13

RESULTS OF TWO-WAY ANALYSIS OF VARIANCE OF BASE COURSE MODULI VALUES

Source of Variance	F	$F_{0.25}$	Significance
Wheel load, C	1.07	1.56	NS
Crack frequency, B	0.06	1.46	NS
Interaction: B x C	0.14	1.56	NS

TABLE 14

COMPARISON OF FALL AND SPRING LAYER DEFLECTIONS, 0.001 IN., 12,000-LB DUAL WHEEL LOAD OR 24,000-LB AXLE LOAD

Station	Subgrade		Subbase		Base		Soil Type	Crack Frequency
	Fall	Spring	Fall	Spring	Fall	Spring		
347	3.7	17.8	2.8	0.9	2.8	14.5	A-6	High
249	4.6	11.1	3.7	9.2	1.0	2.1	A-6	Low
304	4.6	6.3	3.7	12.8	1.0	3.2	A-4	High
275	2.8	7.6	4.6	8.8	0.9	7.4	A-4	Low
460	3.7	8.4	2.8	6.4	1.8	2.9	A-2	High
453	5.6	8.5	0.0	3.7	4.6	4.5	A-2	Low
426	5.6	9.3	4.6	5.0	0.0	4.2	A-1	High
438	4.6	9.3	1.9	3.6	2.8	3.8	A-1	Low

Summary

Figures 4 through 9 summarize the relative modulus values obtained from the fall deflection measurements.

No significant relationship was shown to exist between subgrade modulus values and crack frequency. A significant relationship between crack frequency and subbase moduli was established (at the 0.1 level). It was shown that with 10 percent chance of error, low crack frequency areas have higher subbase modulus values than high crack frequency areas. There appeared to be no relationship between crack frequency and base course moduli. In general, the results indicated that the base course had a higher relative modulus than the subgrade, and the subgrade somewhat higher than the subbase.

The springtime data were obtained only under a 12,000-lb dual wheel. It is to be seen in Table 14 that the macadam base course showed greater relative deflections in the spring than in the fall. The increase in deflection of the base at locations of high frequency was approximately 230 percent as contrasted to the base at low crack frequency locations, 59 percent. The springtime deflections for all the pavement layers were greater than the fall values, the subgrade and base course showing the largest increases. The fall measurements indicated that the subbase contributed significantly to the total deflection, whereas the spring measurements indicated that the base course and subgrade were more significant.

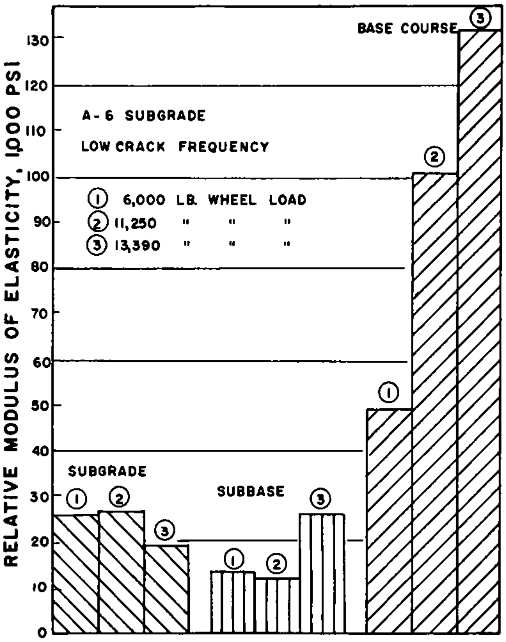

Figure 4. Relative modulus of pavement layers, station 249 (based on fall measurements).

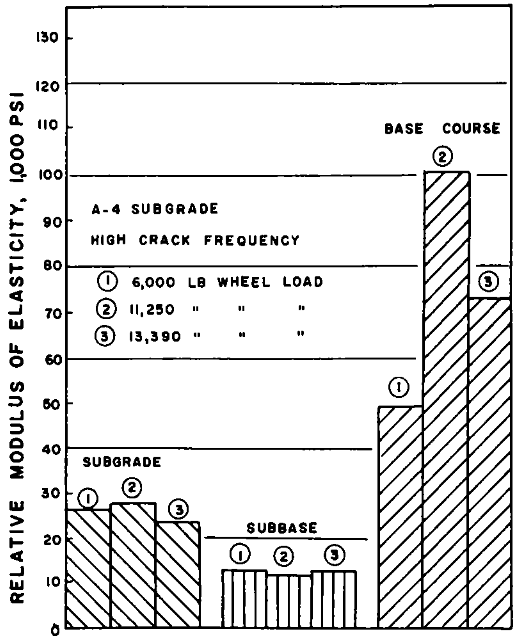

Figure 5. Relative modulus of pavement layers, station 304 (based on fall measurements).

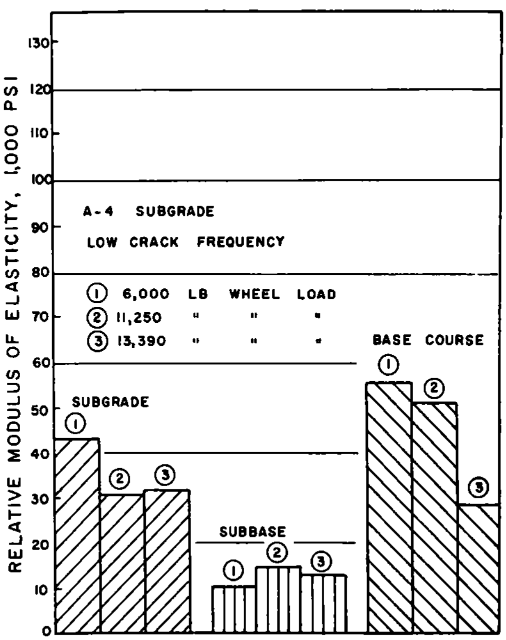

Figure 6. Relative modulus of pavement layers, station 275 (based on fall measurements).

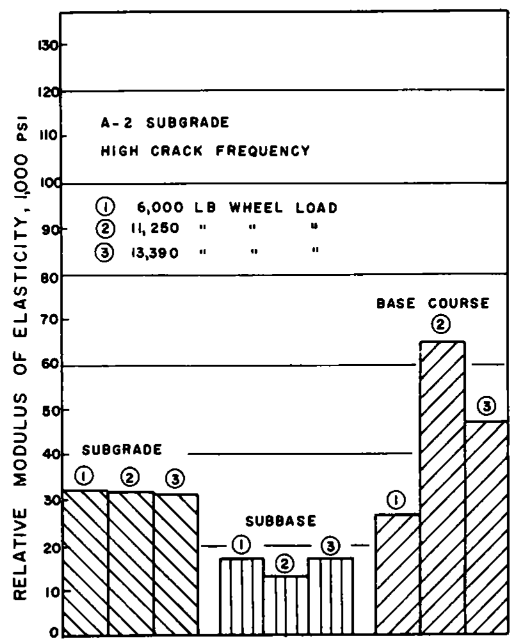

Figure 7. Relative modulus of pavement layers, station 460 (based on fall measurements).

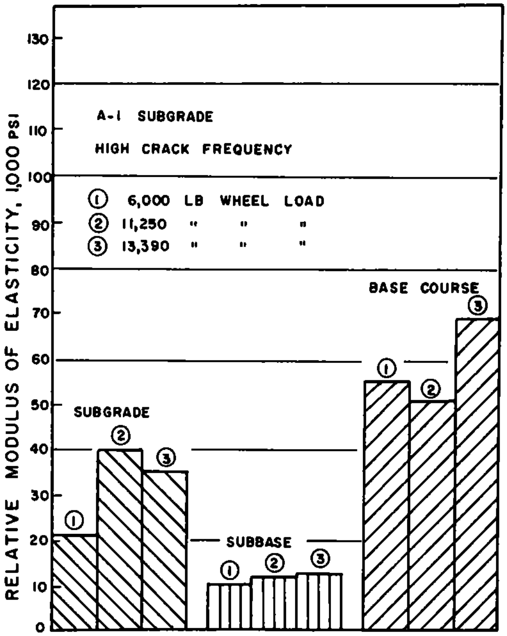

Figure 8. Relative modulus of pavement layers, station 426 (based on fall measurements).

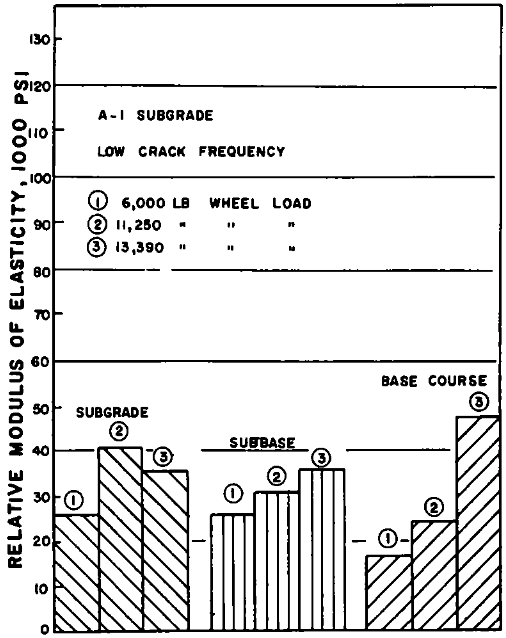

Figure 9. Relative modulus of pavement layers, station 438 (based on fall measurements).

DISCUSSION

Deflection, Rutting and Cracking

As shown in Table 1, pavements built over each of the four subgrade types showed on the average more than 100 lin ft of cracks per station. Because the subgrades ranged from granular materials)A-1-a classification) to silty clay (A-6 classification), it is concluded that low subgrade support was not primarily responsible for the pavement cracking.

More cracks have occurred in the traffic lane than in the passing lane suggesting a relationship between cracking and traffic intensity. The cracking however has progressed since first noticed in 1957 to the point where some sections of passing lane over all the subgrade types are beginning to show as many cracks as sections of the traffic lane; observation seems to indicate that the cracking is continuing at an increasing rate.

The absence of a significant relationship between surface deflection and crack frequency is perhaps the most important finding of the study. In the statistical analysis, the hypothesis was made that no relationship existed between crack frequency and deflection. This hypothesis could not be rejected even at the 0.01 or 0.05 level (Table 3).

Surface deflection, which is the sum of the deformations that occur in all the layers of the pavement structure and the subgrade, is dependent on many factors. Gross load, tire pressures, moisture content of the subgrade, and base, density of all the pavement components, volume change characteristics, and the season of the year, are some of the important factors that affect surface deflection. For example, natural variation in moisture contents from one location to another could cause different deflections, regardless of the cracking that had occurred in the pavement surface.

Another important result of this study is the relationship found between rutting and total deflection. Wheel track rutting reflects the permanent changes in elevation of the pavement layers and subgrade. In this case subgrade type was shown to be a statistically significant factor affecting the amount of rutting. The largest rutting values were associated with the fine-grained subgrades. A good correlation was established between surface deflection and rutting.

Layered System Deflections

The deflection of a component layer of the pavement structure depends on the physical properties of the layer in question and on the stresses imposed upon the layer. Hence, it becomes necessary in the analysis of layer deflections to take into account the stress distribution through the pavement. Use was made of the Boussinesq solution in this analysis to calculate a relative modulus for the pavement layers. The limitations of this analysis should be recognized (that is, homogeneity, elasticity, tire imprint shape, etc.).

Stress computations for the two- and three-layer problem have been proposed by Burmister (12, 13) and others. However, use was made of the homogeneous problem in this study because relative stresses at offset distance from the centerline of each tire of the dual wheel could readily be computed.

Observation of the deflection data in Table 14 and the modulus computations immediately indicate the interesting point that the deflection in the subbase was disproportionately larger in many cases than the deflection in the subgrade.

The subbase as used in this test pavement has two primary functions. First, it is a drainage layer which permits escape of water through the shoulders. Second, it is a transitional layer between the subgrade and base course. To fulfill this second purpose, the subbase ideally must not deform as much, for the same imposed stresses as the subgrade.

The subbase in this road consisted of a relatively clean, cohesionless sand. This satisfied the aforementioned drainage criterion; however, there was difficulty encountered in supporting construction traffic during construction due to the noncohesive nature of the material.

The fact that the subbase moduli were found to be related statistically to crack frequency is considered to be important. Even though the relationship was statistically significant only at the 0.1 level, this information, coupled with the lack of correlation between cracking and the other factors (as evidenced by very low F ratios in the analysis of variance concerning total deflection, subgrade modulus, and base course modulus), indicates that high deformation in the subbase regardless of seasons contributed to the occurrence of surface cracks. However, the spring measurements showed a large increase in deflections on the waterbound macadam base and sizeable increases in the A-6 subgrades and significant increases in the other subgrade types.

Ductility and Penetration Tests

When the deflection holes were drilled in the test pavement, cores of the bituminous layers were removed. Penetration and ductility tests were made on asphalt extracted from these cores. The results of these tests are given in Table 15.

It should be noted that the penetration and ductility values are the lowest for the surface course and the highest for the base course due undoubtedly to oxidation of the upper layer. Both the penetration and ductility values are rather low, particularly for the surface. This reflects conditions of the surface conducive to pavement cracking.

Evaluation of Test Pavement

On the basis of the findings from the crack survey, surface deflection, rutting studies, and the layered system deflection analysis, it is believed that, regardless of the cause of initial cracking, greater than normal deflection in the subbase is an important factor in the progression of cracking that is occurring in the test pavement. This is based on the observation that areas of high crack frequency were associated statistically with the low values of subbase modulus and vice-versa (see Table 10) and crack frequency was not found to be associated with type of subgrade nor with the relative modulus values of either the subgrade or base.

Cracking in a pavement surface can be caused by many factors including (a) shear

TABLE 15

AVERAGE DUCTILITY (cms) AND PENETRATION (0.1 mm) CORE LAYER

Station	Crack Frequency	Core Component					
		Surface		Binder		Base	
		Pen.	Duct.	Pen.	Duct.	Pen.	Duct.
347	High	29	8	39	22	42	34
249	Low	27	7	30	12	35	22
304	High	28	7	35	29	41	39
275	Low	28	6	34	14	41	30
460	High	33	12	40	24	41	36
453	Low	29	7	29	9	32	14
426	High	22	5	28	10	31	19
438	Low	22	6	28	13	33	17

stresses, (b) tensile stresses induced by volume changes in any of the pavement components, (c) tensile stresses caused by deflection of the pavement structure, and (d) bending stresses due to repeated loads which, when few in number are not destructive, but when repeated in numbers commensurate with high traffic volumes can be detrimental.

Shear stresses in the pavement surface can be assumed to be a major factor contributing to cracking where rutting, upheaval outside the loaded area, and other permanent differential settlements are evident. In the case of this road, rutting and cracking were found to be unrelated (Table 6). The cracks that did occur were quite smooth and lacked the evidence of a shear type of failure.

Effectiveness of Evaluation Methods

The procedures used in this study provided a basis on which to develop answers to the question of what was causing the cracking and rutting of the test pavement. However, the problem is of such complexity as to make impossible complete explanations of all behavior of the pavement structure. For example, if the subbase course is partly responsible for the cracking, why do not the cracks resemble map-type cracking more than they do? Also, if the primary cause is shrinkage, why are there so few transverse cracks? Most of the initial cracks were essentially longitudinal with some of the latter cracks tending to be diagonal in direction. A reason for the direction of crack formation could not be obtained from any of the data obtained in this study and at this point can only be presented as conjecture. Inasmuch as one would normally expect shrinkage cracks to be transverse in direction as well as longitudinal and fatigue cracks to be "map cracking," it appears reasonable to assume a combination of causes. It is pertinent to note that because the heaviest loads are on tandem axles, shorter radii of curvature are produced in the transverse direction (across the dual wheels) than in the longitudinal direction. These shorter radii would cause transverse bending stresses greater than longitudinal stresses, thus encouraging longitudinal cracks.

By showing that there was no relationship between total deflection and crack frequency, greater emphasis is placed on the need of measuring deflections in each pavement layer.

Deflection Measurement Procedure

The method of making the surface deflection measurements with the Benkelman beam differs from that used by many engineers. It is common practice to place the probe of the Benkelman beam between the dual wheels at a distance of 4.5 ft under the truck. The truck then pulls forward and initial and maximum dial readings are recorded. This puts the dual wheels initially about 3 ft from the reference feet of the beam. Data obtained during the surface deflection study of this report indicate that in most instances movement of the reference feet would have occurred if this procedure had been used. The procedure used in this study, wherein the dual wheel comes no closer to the feet than 7.5 ft, is recommended.

SUMMARY OF CONCLUSIONS AND RESULTS

Concerning the study of the pavement reported in this paper the following conclusions are drawn:

1. Total deflections, by themselves, were not effective in determining the entire cause of the distress. Surface deflections were correlated with rutting but no correlation was found between cracking and surface deflection.
2. Deflection measurements of the individual layers of a flexible pavement structure were useful in showing the relationship between the pavement cracking and properties of each pavement layer.
3. Determination of relative modulus values were used satisfactorily in an evaluation of the relationship of deflection of one layer to another.
4. The procedure of backing the truck so that the dual wheel just passes over the probe end of the Benkelman beam was found to be preferable to the procedure where the truck begins only 3 ft from the reference feet of the beam.

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Discussion

W. H. CAMPEN, Omaha Testing Laboratories — The authors are to be commended for conducting an extensive investigation in a systematic manner.

The writer has had considerable experience in conducting deflection or deformation tests in connection with test sections on airport runways.

It will be noted in Figure 1 that for each load the total deformation or settlement can be divided into permanent deformation and elastic deformation. The permanent deformation represents consolidation or plastic flow in any or all of the components of the layered system including the subgrade. The elastic deformation or rebound is caused by the compressibility of entrapped air or other gases.

The data in Table 1 show that with any one load, elastic deformation, deflection or rebound, whichever it is called remains practically constant when the application of the load is repeated.

In discussing the paper, first, the significance of the deflection test results depends on the age of the pavement. If the tests are made at the time of construction or soon thereafter, they might reveal both permanent deformation and elastic rebound. On the other hand, if they are made after the pavement has been in use for some time, the tests will indicate only elastic deformation. The reason lies in the fact that pavements which handle satisfactorily the loads for which they were originally designed, will become truly elastic in a comparatively short time.

The writer's second point pertains to the effects of the types of deformation. Consolidation or plastic flow will cause rutting in the traffic lanes regardless of the magnitude of the elastic deflection but does not usually cause cracks of any kind. Cracks of the type that cause break-up are produced by excessive elastic deformation. The excessive deformation bends the surface beyond its flexing limits and, thus, literally shatters the surface.

TABLE 1

EFFECT OF REPETITIVE LOADING ON ELASTICITY

Project	Layer Tested	Plate Area (sq in.)	Load Used (psi)	Elastic Deformation (in)				
				Load Applications				
				1	2	3	4	5
Omaha	12-in. sand gravel base	432	88	0.1475	0 1425	0 1525	0 1400	0.1475
		432	65	0.100	0 095	0 100	0 095	0 100
	12-in. rock base	800	19	0 045	0 045	0 0475	0 0425	0 045
		800	56	0 1225	0 1225	0 1225	0 1275	0 1275
	3½-in. asphaltic concrete wearing surface	800	49	0.220	0 2275	0 220	0 225	0 2275
Dubuque	12-in subbase	432	92	0 1625	0 1675	0 170	0 170	0 1725
		800	62	0 140	0 145	0.1475	0 145	0 145
Waterloo	6-in. base	216	46	0.065	0 0625	0.0675	0 065	0 0675
		216	93	0 1325	0 1275	0 130	0 1375	0 1325

Applying the basic reactions revealed by deflection tests to the results obtained by the authors the writer comes to the following conclusions:

First, that the deflections measured were all elastic because the highway had been in use for two or three years. The fact that the pavement continued to be of service even though it contained large footage of longitudinal cracks shows that the magnitude of the elastic deformation is low.

Second, that the rutting was caused by consolidation in some portion or all of the layered system. It is possible that the bituminous layers alone might have caused all of the rutting.

Third, that the longitudinal cracks are not related to either consolidation or elastic deformation. However, the writer has no explanation for their occurrence.

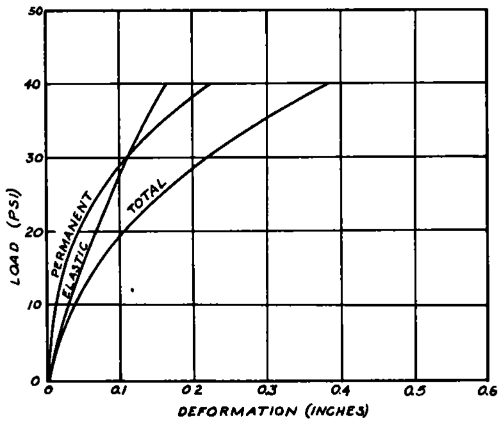

Figure 1. Waterloo Airport test section, 800 sq in. plate on top of subbase.

Fourth, that the apparent correlation of rutting with the magnitude of elastic deformation is no proof that such correlation is sound for the reason that rutting is not related to elastic deformation.

It is evident from the foregoing that deflection tests made after a pavement has been in use for a considerable time do not reveal the cause of rutting. The cause, no doubt, lies in consolidation. This could be proved by making density tests on the components of the layered system and on the subgrade, and then comparing them with densities obtained at the time of construction.

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