**SIGNIFICANCE OF LAYER DEFLECTIONS
IN EVALUATING FLEXIBLE PAVEMENTS**

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Highway
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by

R.D. WALKER

**PURDUE UNIVERSITY
LAFAYETTE INDIANA**

Final Report

SIGNIFICANCE OF LAYER DEFLECTIONS IN EVALUATING FLEXIBLE PAVEMENTS

TO: K. B. Woods, Director
Joint Highway Research Project

January 25, 1961

FROM: H. L. Michael, Assistant Director
Joint Highway Research Project

File: 6-20-3
Project: C-36-52C

A final report entitled, "Significance of Layer Deflections in Evaluating Flexible Pavements" is attached. This report has been authored by Mr. R. D. Walker of our staff under the supervision of Professor E. J. Yoder. Mr. Walker also utilized the report for his thesis in partial fulfillment of the requirements for the Ph.D. degree.

The deflection measurements were made on the U. S. 31 Test Road and twenty-five copies of this report will be transmitted to the Test Road Committee of the State Highway Department for their use.

This report explores techniques which may be useful in evaluating flexible pavements and considerations which will be useful in the development of more effective design methods.

The report is presented to the Board for the record.

Respectfully submitted,

Harold L. Michael
Harold L. Michael
Secretary

HLM:kmc

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by

R. D. Walker
Research Assistant

Joint Highway Research Project
File: 6-20-3
Project: C-36-52C

Purdue University
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January 25, 1961

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ABSTRACT

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Significance of Layer Deflections in Evaluating Flexible Pavements.

Major Professor: Eldon J. Yoder.

To discover what has gone wrong with highways now in use and to find what makes our good highways perform well 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.

The Indiana Test Road, located on U. S. 31 near Columbus, Indiana, is 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 are examined. Total deflections, measured with a Benkelman beam, are analysed in an attempt to establish a relationship between deflection and the cracking that occurred on the test pavement.

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. A device was developed that attached to the

probe end of the Benkelman beam and rested on the reference rod. Measurements were made under rear axle loads of 12,000, 18,000, 22,000 and 27,000 pounds.

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

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

PART I

STATEMENT OF THE PROBLEM

INTRODUCTION

A flexible pavement consists of a relatively thin wearing surface constructed on base and subbase courses. These rest upon the compacted subgrade. By definition, for this study, the thickness of the pavement includes all pavement components above the compacted subgrade. The strength of a flexible pavement is derived from the distribution of load over the subgrade through the surface, base, and subbase courses rather than from the so called "beam action" of the pavement as a whole.

Unlike rigid (portland cement concrete) highway pavements, the thickness design of flexible pavement is influenced to a large extent by type of subgrade. Where, for given traffic conditions, a thickness of ten inches of portland cement concrete might be adequate for every section of a state (indeed, perhaps for every section of the country), the required thickness of a flexible pavement might ideally be changed every mile of a highway. It is difficult to establish what an adequate thickness is, and which of several adequate combinations of layer thicknesses will be the least costly in the long run.

However, before an effective design method can be developed for flexible pavements, a method must be found to evaluate properly our existing highways. To be able to discover what has gone wrong with some highways now in use and to find what makes our good highways perform well is essential. It is not enough to know that certain pavement types and thicknesses have produced good or bad results; we must also know why they have done so.

Many states are now trying to evaluate their existing flexible pavements by one or more methods, among the more popular of which is one based on deflection measurements. Towards this end, the Benkelman beam, which was developed at the WASHO Road Test early in 1953, has become very popular as a simple portable device for measuring total pavement deflections under actual truck loadings.

In general, analysis of deflection data in pavement evaluation is aimed at establishing critical maximum deflections for pavements that have given good service. Thus, if a pavement deflects more than the critical amount, poor performance can be expected.

Although progress has been made in this direction, much remains to be learned, e.g. what represents pavement failure? One who is interested in reading an excellent work in this area is referred to a paper by Carey and Irick (1)*. Along this same line of questioning, what are the symptoms a highway pavement might show before it can be considered to be a failed pavement or to be a pavement with poor riding qualities? Can these symptoms also be related to some criteria such as deflection? These factors represent essential considerations in progress towards an effective method of pavement design.

In this thesis, techniques are explored that might be useful in evaluating existing pavements and answering some of the above questions. Use is made of the Benkelman beam for measuring not only total deflections, but also deflections of individual pavement layers. Also the importance of rutting, pavement cracking, and soil type is discussed. The role of elastic behavior and modulus of elasticity is considered as a tool for analysing pavement behavior.

* Numbers in parentheses refer to references listed in Bibliography.

FLEXIBLE PAVEMENT BEHAVIOR AND DEFLECTIONS

Flexible Pavement Behavior

The greatest application of deflection criteria has dealt with the elimination of pavement failure resulting from excessive plastic deformation of the underlying materials. These failures occur when permanent deformation of the soil is of such a magnitude as to create an uneven riding surface and to cause cracking of the surfacing material. Current pavement design methods have greatly reduced these failures; therefore, fatigue type failures due to recoverable deformations of the pavement and its foundation have grown in importance. (2)(3)

These recoverable deformations have in the past been assumed to be small and normally insignificant in most, if not all, methods of design. For this reason, these deformations might be used to explain some of the pavement failures which occur when established design methods are used. (2)(3)

During the past few years, the concept has developed that under the single application of a moving wheel load on a pavement surface, nearly elastic behavior exists. Assuming that recovery of volume change and behavior in accordance with Hooke's Law are major requirements for elastic action, the only apparent non-elastic effect with reference to a single load is that a measurable amount of time is required for complete rebound. Baker and Papazian (4) point out that "perhaps one of the reasons for the little use made of the apparent elastic behavior of flexible pavements is their obviously non-elastic ultimate behavior."

Hveem (5) has studied pavement distress and deflections rather extensively in California. He believes that the most severe stresses in the pavement are associated with compression and rebound in the upper layers of the embankment which may produce sharper bending and consequently greater stress in the pavement slab than compressions in the lower layers. A moving wheel load causes changes in state of stress in every portion of the pavement in the wheel path. The magnitude of vertical deflection is not of itself completely significant, as the tendency to break or rupture the pavement will depend primarily upon the sharpness of the arc or curvature of the pavement surface.

Jeuffroy (6) suggests that modern flexible pavements with thick surfaces constitute a transition between the classic flexible pavement and portland cement concrete pavements. Cracking in these thicker surfaces is due to flexure and fatigue. Concerning open longitudinal cracks in these pavements, Jeuffroy says these cracks can be caused by heavy loads causing negative bending stresses on planes parallel to the road axis.

Hveem (5) makes an interesting comment concerning pavement thickness: "A thin pavement may serve quite well over resilient soils where a heavier, stronger, but more rigid or brittle type will crack and perhaps show other signs of failure." Of course, many thin pavements fail due to causes other than fatigue or bending.

Pavement Deflection Measurements

Highway engineers have been measuring pavement deflections under various kinds of load for many years. Probably the most common type of measurement made has been in the plate load test where settlement is measured as the load on a circular steel plate is steadily increased. This test does not use a loading procedure that can be compared with the loads produced by actual traffic.

Hveem (5), in 1938, was one of the first to try to measure flexible pavement deflections under vehicular load. He used a General Electric travel gauge to measure deflections of pavements under rapidly moving wheel loads. This gauge was installed in the pavement surface. A cased reference rod was installed at various depths, the top of the rod moving through the gauge producing an electrical impulse. Other investigators (7) have used LVDT (linear variable differential transformers) which operate in a similar fashion. Baker (8) describes a photogrammetric technique for measuring pavement surface deflections under dynamic loading. These three methods all have advantages and disadvantages; not the least of the disadvantages is the cost, difficulty of installation, and complexity of the equipment used.

Development of Benkelman Beam

A. C. Benkelman, early in 1953, while assigned by the Bureau of Public Roads to the WASHO Road Test, developed a lever-arm type of beam device that was capable of recording deflection at the road surface. Since this device was used to obtain most of the data for this thesis, the remaining discussion in this section will concern its description and use.

Also called a lever arm deflection indicator, the Benkelman beam is simply a long lever arm with a three point support pivoted fulcrum. As originally used at the WASHO Road Test, the lever arm or probe extended between the tires of the dual wheel of a loaded truck a distance of four feet, five inches beyond the dual wheel axle under the truck. The truck moved forward at creep speed, and readings were taken before the truck moved, at maximum deflection, and after the truck had pulled beyond the influence of the beam probe. By the completion of the WASHO Road Test, the Benkelman beam was perfected and proved to a point where it became a generally accepted device for measuring surface deflections of pavements. (7)(9)(10)(11)(12)

Use of Benkelman Beam

The Canadian Good Roads Association was perhaps the first national organization to support extensive use of the Benkelman beam. They adopted a procedure different from that of the WASHO Road Test, because they felt that by extending the probe under the truck, both the reference feet of the beam and the probe end were often affected by the close proximity of the wheel load before the test began. The CGRA procedure is to place the probe between the tires directly under the axle, and then to drive the truck away at creep speed. They have also standardized the wheel load at 9,000 pounds and tire pressure at 80 psi (this load and tire pressure give nearly circular tire imprints). By placing the probe directly under the axle, the reference feet were eight feet away from the load, and the probe end was in position of maximum deflection. In all but a few rare cases, it was found that

the eight foot distance was sufficient to prevent the reference feet from being influenced by the load. (10)

Typical use of the Benkelman beam in an extensive pavement evaluation program in Canada is described by Davis, McLeod, and Bliss. (13) Ten outer wheel path deflection measurements were taken; location of each was chosen at random within a specified length of pavement. The deflection value that was exceeded by only ten percent of the pavement length was used as the best indicator for pavement performance. The authors pointed out that the average deflection value might work as well only if the standard deviations of the measurements of each test section were the same. Their experience has shown that this was not the case.

Huculak (14) of Canada describes the use of the Benkelman beam in the evaluation of a failed pavement. About 200 feet of the Banff-Jasper Highway was showing serious distress in the form of alligatoring and pitting. The beam was used to evaluate the entire area to show exactly where reconstruction was to be accomplished. They found that the degree of surface deterioration agreed with the magnitude of the surface deflection as follows:

1. The pavement had pitted out in areas where the deflections were greater than 0.275 inches.
2. Where deflections were greater than 0.135 inches, the pavement had reached the alligatoring stage.
3. At deflections of 0.090 inches and greater there was noticeable rutting.

Using these values as a guide it was decided to reconstruct all areas showing a deflection of greater than 0.045 inches.

Another use of the Benkelman beam is described by Sebastyan (15) of Canada. He describes work which correlates Benkelman beam deflection measurements with plate load tests. He compares the secant modulus obtained from plate load tests with the secant modulus obtained by beam deflection. The definitions of the secant modulus were as follows:

$$\tan \alpha = \text{secant modulus of plate test} = A/B$$

A = load on a 30 inch diameter plate at 10 repetitions
at 0.5 inch deflection

B = 0.5 inch deflection

$$\tan \beta = \text{secant modulus of beam deflection test} = A'/B'$$

A' = 9,000 pound wheel load at 80 psi tire pressure

B' = deflection measurement in inches

Sebastyan also discusses vertical stress distribution while comparing distribution of stresses under a plate with those under a tire.

Williams and Lee (16) have reported a study which consisted essentially of a series of load deflection tests in the spring and fall of each year. Using the WASHO Road Test Benkelman beam procedure, they attempted to correlate the deflection data with some of the variables known to affect pavement behavior. They found that their springtime measurements, using a 11,200 pound dual wheel, were approximately 0.035 inches, while in the fall the measurements were about 0.025 inches. They found no general correlation between deflection and pavement distress.

Some of the most comprehensive work using the Benkelman beam as well as other measuring devices (LVDT's) is being accomplished at the AASHO Road Test in Ottawa, Illinois. Since the testing is not complete at this writing, results have not been published.

WASHO Road Test deflection results (17) have been discussed by Benkelman (18) with his conclusions concerning for the most part comparisons of pavement edge and center deflections. However, it is interesting to note the magnitude of deflections that were obtained. With four inch asphaltic concrete surface on base thicknesses ranging from two to eighteen inches, outer wheel path deflections ranged from 0.11 inches for the two inch base to 0.025 inches for the eighteen inch base. Loads ranged up to 40,000 pounds on a tandem axle.

PURPOSE AND SCOPE

The purpose of this thesis was to develop and study a practical step by step procedure for evaluating the cause of distress in a flexible layered system type of pavement and to validate its usefulness. Emphasis was placed on determining whether or not total pavement deflections are effective in evaluating pavement distress, this being a design criterion that is used by many engineers.

The particular case used for this investigation was the Indiana Test Road, located on U. S. 31 near Columbus, Indiana. This road was built under closely controlled conditions over four major subgrade soil types. More important, it has two major defects, rutting and cracking. To determine from what these defects result was the object of this study.

Theory has indicated that deflection is a function of load and elastic modulus, E (18). It is known that pavement rutting is related to plastic deformation and that fatigue is associated with elastic deformation. Therefore, analysis of cracking, rutting, and deflections were used to evaluate the Test Road.

To date, total deflections have been used in an arbitrary manner, such as setting 0.05 inch deflection as demarcation between good and poor pavements. Whether or not this type of criteria is valid was investigated. To this end a study of individual pavement layer deflections was made. As a means of evaluation of layer deflections, relative moduli values were used. An evaluation of procedures used is presented and conclusions are drawn as to their effectiveness.

INDIANA TEST ROAD - U.S. ROUTE 31 (19)

In February 1951, the Indiana State Senate charged the State Highway Department with the responsibility of conducting an experiment to determine the relative merits of flexible and rigid pavements. In compliance the department selected U.S. Route 31, from 1.8 miles northwest of Columbus to 21.8 miles north of the Bartholomew-Johnson County line, as the pavement comparison site.

This area is along the southern border of the Wisconsin drift and is crossed by the Shelbyville and Champaign moraines. The entire length of the project parallels the border of Wisconsin drift.

There are approximately 4.3 miles of flexible pavement in the southbound direction of this dual lane construction and 2.8 miles in the northbound direction. The road was opened to traffic on December 11, 1953.

Design

The flexible pavement design was based upon a combination of the Corps of Engineers CBR method and the Group Index method. Soil types ranged from A-1-a sands and gravels to a plastic A-6 clay (AASHTO soil classification). The basic design of the pavement structure was as follows:

- 1 inch - asphaltic concrete surface course
- 1½ inches - asphaltic concrete binder course
- 2½ inches - asphaltic concrete base course
- 8 inches - waterbound macadam base course

5 inches (inside edge) to 8 inches (outside edge) - open-graded
drained granular subbase course

Construction

The subgrade, after compaction to 100 percent of the maximum standard AASHO value, received at least two coverages with a heavy pneumatic roller (20-35 tons gross load and 50-70 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 following initial compaction to 100 percent of maximum standard AASHO density. Since the subbase lacked cohesion, the heavy pneumatic compactor could not be used. The multiple shoe vibrator was not employed until the top two and one-half inches were mixed with 70 pounds per square yard 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.

A 60-70 penetration asphalt was used in all three bituminous concrete layers in percentages of 4.5, 5, and 6 percent for the base, binder, and surface courses respectively.

Observance of Cracking and Rutting

Early in 1957, longitudinal cracking began to appear in several locations of the flexible pavement. The cracking was generally associated with both wheel tracks. Where the cracking was severe, some transverse cracking was apparent. Examples of the cracking are illustrated in Figure 1. Wheel track rutting was observed at the same time

FIG.1 EXAMPLE OF PAVEMENT
CRACKING

as the cracking. In general the rutting did not exceed 0.5 inches, but it occurred throughout the road.

PART II

CRACKING, RUTTING, AND SURFACE DEFLECTION STUDY

Cracking Study

Complete crack data, in the form of scale maps of the test road, have been available through "Test Road Committee Memoranda" published by the State Highway Department of Indiana. The most recent data, obtained in March 1960, were used in this study (20).

Cracking and Soil Type

In Table 1 are tabulated the average linear feet of cracks per station (100 feet) for the four basic soil types* found under the flexible pavement test sections. All 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	Number Stations	Linear Ft. Cracks	Average Linear Ft. Cracks per Station
A-1	39	4,814	123
A-2	53	5,863	111
A-4	177	23,901	135
<u>A-6</u>	<u>109</u>	<u>11,138</u>	<u>102</u>
All	378	45,716	121

* Highway Research Board classification used throughout thesis.

Table 1 shows that more cracks have 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 Relationships

As indicated by Table 2, the greater number of cracks occurred in the traffic lane as compared to the passing lane.

Table 2. Relationship Between Lane Type and Cracking

Soil Type Lane	A-1		A-2		A-4		A-6	
	Traf- fic	Pass- ing	Traf- fic	Pass- ing	Traf- fic	Pass- ing	Traf- fic	Pass- ing
No. of Stations	39	39	53	53	177	177	109	109
Linear ft. Cracks	2645	2169	4614	1249	117683	6218	8708	2430
Ft. Cracks/Station	68	56	87	24	100	35	80	22
Difference	✓12	-	✓63	-	✓65	-	✓58	-
% Cracks Traffic Lane	55		79		74		78	

Table 3 presents the data according to wheel path. There appears to be little relationship between cracking and location of wheel path. This may be partly due to the examiner's inability to assign properly a crack to a wheel path. Also, many of the cracks are not actually associated with the wheel paths.

Another factor that may have an effect on pavement performance is pavement surface thickness (bituminous concrete surface and base courses combined). Table 4 summarizes the cracking data with relation to the

Table 3. Relationship Between Wheel Path and Cracking

Soil Type	Lane	Wheel Path	No. Stations	Cracks ft.	Cracks ft./sta.	% Cracks Inner Wheel Path
A-1	Traffic	Outer	39	1660	43	37
		Inner	39	985	25	
	Passing	Outer	39	586	15	73
		Inner	39	1583	41	
A-2	Traffic	Outer	53	1966	37	57
		Inner	53	2648	50	
	Passing	Outer	53	936	18	25
		Inner	53	313	6	
A-4	Traffic	Outer	177	10174	57	42
		Inner	177	7509	42	
	Passing	Outer	177	3497	20	44
		Inner	177	2721	15	
A-6	Traffic	Outer	109	5098	47	41
		Inner	109	3610	33	
	Passing	Outer	109	1194	11	51
		Inner	109	1236	11	

Table 4. Relationship Between Pavement Surface Thickness and Cracking

Soil Type	Lane	Surface Thickness inches	No. Stations	Cracks ft.	Cracks ft./sta.	% Cracks 5' Surface
A-1	Traffic	5+	13	418	32	27
		5-	26	2227	87	
	Passing	5+	15	924	62	54
		5-	24	1245	52	
A-2	Traffic	5+	21	1653	79	46
		5-	32	2961	93	
	Passing	5+	26	687	26	55
		5-	27	562	21	
A-4	Traffic	5+	64	5814	91	47
		5-	114	11869	104	
	Passing	5+	61	2451	40	56
		5-	117	3767	32	
A-6	Traffic	5+	15	1018	68	45
		5-	94	7690	82	
	Passing	5+	32	971	30	61
		5-	77	1459	19	

design thickness of the bituminous concrete. At least as far as the traffic lane is concerned, fewer cracks have appeared in sections thicker than five inches than in the sections less than five inches thick. It is readily seen that this relationship does not hold true in the passing lane.

Surface Deflection Study

Description

During the first week of June 1959 and the second week of May 1960, personnel from the Joint Highway Research Project at Purdue University and from the State Highway Department of Indiana secured Benkelman beam deflection measurements 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 illustrated in Figure 2. The variables selected were 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. One showing 25 feet or less lineal feet of cracking was classified as having low crack frequency. Since the nominal paving thickness was five inches, this was used as the demarcation line for the surface thickness variable. The sites were also selected on the basis of soil type.

Special Testing Equipment

The deflection measurements were made using Benkelman beams, shown in Figure 3. During each testing program, one beam was supplied by Purdue University and one by the State Highway Department. The test vehicle was a State Highway truck with an 18,000 pound rear axle load.

Figure 2. Design of Experiment for Benkelman Beam Deflection Measurements

FIG. 3 BENKELMAN BEAM IN ACTION

Testing Procedures

At each location, deflection measurements were taken at five basic positions as illustrated in Figure 4. These positions were as follows:

1. one inch outside the outside dual wheel
2. between the tires of the outside dual wheel
3. at the centerline of the truck
4. between the tires of the inside dual wheel
5. one inch outside of the inside dual wheel.

In 1959, the Purdue Benkelman beam was responsible for all the readings at positions 2, 4, and 5 and in 1960 positions 2, 3, and 4. The State Highway Department beam was used at positions 1 and 3 and 1 and 5 in 1959 and 1960 respectively.

Readings were taken when the rear axle of the truck was at each foot mark beginning at ten feet from the particular location. The truck was backed to the zero line so that its rear axle passed beyond the zero foot line approximately three inches, and then it was moved forward. Readings were taken during both the loading and unloading cycles.

Having an experienced and patient truck driver was of great importance on this project, because a large degree of skill was required to back the truck successfully over the Benkelman beam probe without hitting it.

Test Results

In presenting the deflection data, emphasis is placed on "between the dual wheel" readings (positions 2 and 4), because these positions inherently permitted greater precision in locating the probe of the Benkelman beam. Positions 1 and 5 provided less precise data, because the

**FIG. 4 DEFLECTION MEASUREMENT LOCATIONS
AND TIRE DATA**

distance of the beam probe from the side of the truck tire varied more than when the probe was between the tires of the dual wheel. In order to keep the data symmetrical, that is an equal number of locations in the variables of soil type, lane, pavement performance, and pavement surface thickness, emphasis in the analysis is placed on the 32 locations previously described.

The basic deflection data obtained in the field are available from the Indiana State Highway Department (20). Plots of these data in the form of longitudinal and transverse deflection contours have been drawn, but they are not included here because of their large number. Examples of these curves are shown in Figures 5 and 6. It should be noted that the loading and unloading portions of the curves in these figures are not identical. This is probably due to the time effect of the load and to the fact that the pavement and underlying material are not homogeneous. These and other factors result in a somewhat non-elastic response.

The deflection data in terms of their relationship to soil type, lane, crack frequency, and surface thickness are summarized in Appendix A. Table 5 presents the combined 1959 and 1960 inner and outer wheel path data.

Statistical Analysis

A four-way classification analysis of variance was performed on various aspects of the deflection data. The details of the method and model used may be found in Appendix B. Significance of a factor was determined by variance ratios or "F" tests at 0.05 and 0.01 levels. For the effect of soil type on deflection, the Tukey method for determining a studentized range allowance for a set of means was applied (21).

FIG. 5 TYPICAL LONGITUDINAL DEFLECTION CURVES

FIG.6 TYPICAL TRANSVERSE DEFLECTION CURVES

Table 5. Summary of Total Deflection for 1959 and 1960 Data, Inner and Outer Wheel Path

Soil Type Lane	Total Deflection, 0.001 inch								
	A-1		A-2		A-4		A-6		
	Traffic	Passing	Traffic	Passing	Traffic	Passing	Traffic	Passing	
Low Crack Frequency	Thick Surface	15 18 14 16	15 18 14 16	9 18 11 15	20 32 18 20	13 17 14 18	24 32 20 25	14 22 12 18	127 174 115 143
									559
									1200
	Thin Surface	21 20 20 20	13 13 11 14	16 19 10 20	30 30 25 24	24 36 19 26	22 21 18 19	17 21 16 22	163 179 135 164
High Crack Frequency	Thick Surface	11 10 13 12	12 14 15 12	11 12 13 18	26 26 20 23	10 16 10 22	18 19 14 19	13 30 17 17	118 153 117 145
									533
									1144
	Thin Surface	14 15 15 15	16 11 13 13	12 13 10 8	24 24 25 24	20 22 18 24	34 34 34 34	15 17 14 23	153 154 146 158
Totals		249	220	215	391	309	387	288	2344
		469	500	700	675				

Table 6 summarizes the results of the analysis of variance of the data presented in Table 5.

The statistical analysis shows that soil type, lane, and surface 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 did not prove to be related significantly to crack frequency. The interactions shown to be significant 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.

In order to determine the significant differences between deflections of different soil types, the previously mentioned Tukey's method was applied. The average deflection values for the four soil types were:

$$A-1, 469/32 = 14.6$$

$$A-2, 500/32 = 15.6$$

$$A-4, 700/32 = 21.9$$

$$A-6, 675/32 = 21.1$$

The statistical data required for the Tukey analysis are:

$$EMS \text{ (error mean square)} = 12$$

$$n = df \text{ (degrees of freedom)} = 96$$

$$g = \text{(number of data groups)} = 4$$

$$m \text{ (number of items in group)} = 32$$

$$q \text{ (studentized range coefficient for } g = 4 \text{ and } n = 96 \text{ at the .05 level)} = 3.71$$

$$RF \text{ (range factor)} = q\sqrt{EMS/m} = 3.71\sqrt{12/32} = 2.3$$

Table 6. Four-Way Analysis of Variance Results for 1959 and 1960
Deflection Data, Inner and Outer Wheel Path

Source of Variance	F	F.05	F.01	Significance
Soil Type, A	36.40	2.71	4.03	.01 level
Lane, B	50.98	3.95	6.96	.01 level
Crack Frequency, C	2.00	3.95	6.96	NS
Surface Thickness, D	16.66	3.95	6.96	.01 level
Interactions: A x B	2.33	2.71	4.03	NS
A x C	2.92	2.71	4.03	.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	.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	.05 level
A x B x D	4.83	2.71	4.03	.01 level
A x C x D	7.00	2.71	4.03	.01 level
A x B x C x D	7.91	2.71	4.03	.01 level

Comparing the range factor to the ranges between the means of the four soil types:

A-1 and A-2; $15.6 - 14.6 = 1.0 < 2.3$, not significant

A-2 and A-4; $21.9 - 15.6 = 6.3 > 2.3$, significant

A-4 and A-6; $21.9 - 21.1 = 0.8 < 2.3$, not significant

The above shows that real differences exist in deflection measurements made on coarse and fine-grained soils, but differences between A-1 and A-2 or A-4 and A-6 are not significant. It can be stated that deflections on A-1 and A-2 subgrades are lower than those on A-4 and A-6 subgrades.

Wheel Track Rutting Study

During the first week of August 1959, a transverse profilometer constructed by the Bureau of Materials and Tests of the State Highway Department was used to obtain the wheel track rutting measurements at the same 48 locations used in the surface deflection study. Complete construction and operating instructions may be found in the Test Road Committee Memoranda of the State Highway Department of Indiana (20). The profilometer is shown in Figure 7.

Test Results and Analysis

The basic rutting data are available from the Indiana State Highway Department (20) in the form of transverse profile curves for each of the 48 locations, an example of which is shown in Figure 8. A rutting value was determined for each location by determining the maximum difference in elevation for that location in inches.

FIG. 7 TRANSVERSE PROFILE MEASURE-
MENT PROFILOMETER

FIG. 8 TRANSVERSE PROFILES

Table 7 presents a summary of the wheel track rutting data. Table 8 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 were significantly affecting the rutting data. The average rutting values for the four soil types were:

$$A-1, 288/16 = 18.0$$

$$A-2, 429/16 = 26.8$$

$$A-4, 524/16 = 32.8$$

$$A-6, 555/16 = 34.7$$

The statistical data required for the Tukey analysis are:

$$EMS \text{ (error mean square)} = 115$$

$$n = df \text{ (degrees of freedom)} = 32$$

$$g = \text{(number of data groups)} = 4$$

$$m = \text{(number of items in groups)} = 16$$

$$q \text{ (studentized range coefficient for } g = 4 \text{ and } n = 32$$

$$\text{at the } 0.05 \text{ level)} = 3.84$$

$$RF \text{ (range factor)} = q\sqrt{EMS/m} = 3.84\sqrt{115/16} = 14.3$$

Applying the range factor to the ranges of the means of the four soil types:

Table 7. Summary of Inner and Outer Wheel Track Rutting Data

Soil Type Lane	Rutting, 0.01 inch										
	A-1		A-2		A-4		A-6		Totals		
	Traffic Passing		Traffic Passing		Traffic Passing		Traffic Passing				
Low Crack Frequency	Thick Surface	13	11	27	22	36	33	62	25	229	439
		33	16	28	22	21	31	33	26	210	
	Thin Surface	40	11	44	12	44	24	53	18	246	412
		27	10	26	17	30	16	24	16	166	
High Crack Frequency	Thick Surface	16	11	34	21	62	23	60	25	252	449
		12	12	33	23	38	24	31	24	197	
	Thin Surface	28	19	41	31	61	22	79	21	302	496
		15	14	22	26	35	24	40	18	194	
Totals		184	104	255	174	327	197	382	173	1796	
		288		429		524		555			

Table 8. Four-Way Analysis of Variance Results for Inner and Outer Wheel Track Rutting Data

Source of Variance	F	F. _{.05}	F. _{.01}	Significance
Soil Type, A	7.83	2.92	4.51	.01 level
Lane, B	34.00	4.17	7.56	.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

A-1 and A-2, $26.8 - 18.0 = 8.8 < 14.3$ not significant

A-1 and A-4, $32.8 - 18.0 = 14.8 > 14.3$ significant

A-2 and A-4, $32.8 - 26.8 = 6.0 < 14.3$ not significant

A-2 and A-6, $34.7 - 26.8 = 7.9 < 14.3$ not significant

A-4 and A-6, $34.7 - 32.8 = 1.9 < 14.3$ not significant

Real differences exist in rutting measurements made on A-1 subgrades and the fine-grained subgrades. The A-2 soil group could not be established as performing differently from any of the three other soil groups. Thus, rutting measurements of pavement on A-1 subgrades are significantly lower than those of pavements on A-4 and A-6 subgrades.

Relationship Between Deflection and Rutting

As can be seen by Figure 9, there is a good relationship between maximum deflection and wheel track rutting. This is due to the fact that both are related to subgrade soil type. The fitted line of Figure 9 was obtained by the method of least squares (21). It should be noted, however, that the points in this figure represent average values, and for one soil type (A-4) the relationship is not as good as for the other soil types.

Summary

At this point the most important results of this section of the report are summarized.

Cracking

On the basis of number of cracks per 100 feet, the greatest number of cracks occur in the traffic lane and on A-4 (silt) type subgrades.

FIG. 9 MAXIMUM DEFLECTION VS RUTTING

Fewer cracks occur in pavement built on A-6 (clay) type subgrades. However, with pavement over all subgrade types exhibiting more than 100 feet of cracks, on the average, per station, it is safe to say that subgrade type is not the major factor causing the pavement cracking.

The difference in the number of cracks occurring in the traffic and passing lanes is clear. Obviously, the greater traffic volumes on the traffic lane have caused a faster rate of crack progression in that lane than in the passing lane.

When considering the traffic lane only, fewer cracks occur where the surface courses (bituminous base, binder, and surface) are of design thickness or greater than when they are below design thickness. This relationship does not hold true for the passing lane; in fact, it is reversed. With the exception of traffic lane, A-1 subgrade, which showed only 27 percent cracks on five-plus inch thick pavement surface, all cases showed percentages near 50 percent. Therefore it must be concluded that there is no clear relationship between pavement surface thickness and cracking.

The following statements summarize the cracking study:

1. Cracking is not shown to be related to subgrade type.
2. More cracking occurs in the traffic lane than in the passing lane, probably due to greater traffic volumes on the traffic lane.
3. Pavement surface thickness is not shown to be related to cracking.

Surface Deflection and Rutting

Surface deflection data were affected by subgrade type, lane, and surface thickness. More important, there was no statistically significant effect of pavement crack frequency on these total deflection values. As with the surface deflection data, the rutting data were affected by subgrade type and lane, and not by crack frequency. Unlike the surface deflection data, the rutting data were unaffected by pavement surface thickness. The fact that both rutting and deflection values were affected by soil type explains the relatively good relationship found between deflection and rutting. In summary, the following statements can be made:

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 relationship exists between rutting and total deflection.

PART III

LAYERED SYSTEM DEFLECTION STUDY

Introduction

The next step in the investigation dealt with is 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 distress could be established. Through the use of Boussinesq stress distribution theory, relative moduli values, E , could be derived from the deflection data. The term relative modulus is used instead of elastic modulus, since the pavement structure is imperfectly elastic. This is more fully discussed in a later section. It was thought that the relationship between relative E values of the pavement layers might be the most useful tool in establishing the cause of the existing pavement distress. It should be noted that it was necessary to assume that material under the cracked pavement underwent the same relative changes, with time, as the material under 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, which is diagrammed in Figure 10, utilizes four-inch diameter holes, with pipe casing and steel reference rods of varying lengths. Also an extension device was developed to fit on the probe end of the Benkelman beam to enable it to be attached to the rods inside the hole. The difference in deflections obtained from two adjacent holes yields the actual settlement of the

FIG. 10 USE OF BENKELMAN BEAM
FOR LAYERED SYSTEM
DEFLECTION STUDY

layer between the reference plates of the two holes.

It was considered impractical to install a complete set of holes and reference rods at all 32 sites of the previously discussed experiment; therefore, only eight sites were selected. The variables of lane location and pavement surface thicknesses were investigated only at a single level. It is recognized these both influence the magnitude of surface deflection, but the passing lane was chosen because it more nearly approaches the original condition of the pavement, and surface thicknesses as close as possible to the nominal design of five inches were chosen in order to eliminate the effect of abnormally thick or thin surfaces. 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.

This phase of the project, as with the others, was performed by personnel from the Joint Highway Research Project in cooperation with personnel from the State Highway Department of Indiana. All of the field work was completed during the month of September 1960.

Location and Installation of Test Holes

Location

The eight sites selected for this study conform to the pattern of variables illustrated in Figure 11. At each site, four holes and a total of five measurement locations (one surface measurement and four measurements at different depths) were planned. An attempt was made to

Figure 11. Design of Experiment for Layered System Deflection Study

space the holes as close to each other as possible and yet not have the spacing so close as to affect the deflection data. To meet this requirement, a spacing of six feet was selected.

The five measurement locations were in the inner wheel path of the eight sites as follows:

1. surface
2. hole number 1 - through the surface to top of waterbound macadam
3. hole number 2 - through the surface and waterbound macadam to top of subbase
4. hole number 3 - through the surface, waterbound macadam, and subbase to top of subgrade
5. hole number 4 - through the surface, waterbound macadam, subbase, and first five feet of subgrade.

At all eight sites the measurement locations were laid out from south to north.

Installation

Holes were drilled through the surface and waterbound macadam by a portable core driller equipped with a four and one-half inch outside diameter diamond bit. The drill was equipped with a water pump which insured an adequate water supply during the coring of the waterbound macadam. A post hole digger was used to penetrate the subbase and the subgrade material. All subbase drilling was accomplished with a four inch bit. At two sites, the four inch post hole digger was used to advance the hole five feet into the subgrade. In these cases, the hole

was encased with pipe for its entire depth. At four sites, the four-inch bit was used for the first foot of the subgrade and then a two and one-half inch bit was used for the remaining four feet. Here, only the first foot was encased. At the two remaining sites, neither the four nor the two and one-half inch bit could penetrate more than one foot into the subgrade because of its dense granular nature.

Electrical conduit with an outside diameter of four inches was used as hole casing. The pipe was cut in lengths corresponding to the four hole depths. Lengths were based on maximum hole depths, and the pipes were scored at one inch intervals to enable cutting in the field to exact lengths. Voids between the pipe and the sides of the hole were filled with plaster of Paris. For holes number 1, 2, and 3, aluminum foil was wrapped around the bottom two inches of the pipe exterior and held in place with drafting tape. The sole purpose of this was to cause as tight a fit as possible at the bottom of the hole and thus prevent plaster of Paris from leaking under the pipe. This procedure was used in all but the first set of holes installed (Station 304, F-3) and as will be shown later, the results were adversely affected for that one location. The entire pipe, inside and out, and the foil and tape, were liberally coated with oil before insertion into the hole. This was to prevent bond of the plaster of Paris with the pipe.

Three-quarter inch hexagonal cold finished steel rod was used for reference rods. The description of these rods as well as of the assembly that connected the rods to the Benkelman beam is given in Figure 12. A small amount of plaster of Paris paste was placed in the bottom of the

hole to provide a solid level foundation for the reference rod plate.

Hole caps (Figure 13) made of sixteen guage sheet metal were fabricated. Hardened steel nails were used to secure the hole caps.

Test Procedures

Equipment

The basic equipment used in the layered system deflection study was the same as used in the May 1960 deflection study previously described with the exception of the extension device which was attached to the probe of the Benkelman beam. The only additional equipment consisted of four "Loadometer" devices, used for weighing the truck. One of the "Loadometers" is shown in Figure 14.

Truck Loading

Four different truck loadings were used as follows:

1. 6,000 pounds, right rear dual wheel load
2. 9,250 pounds, right rear dual wheel load
3. 11,250 pounds, right rear dual wheel load
4. 13,390 pounds, right rear dual wheel load

The Loadometers were used as shown in Figure 15 to make certain the load was distributed equally over the rear axle.

Deflection Measurements

The basic procedures were the same as those used in the previously described studies. However, there were some differences. While one beam was being used to record deflections between the tires of the right dual wheel of the truck, the second beam was used to make check

FIG 13 DEFLECTION HOLE CAP
INDIANA TEST ROAD

FIG. 14 "LOADOMETER" TRUCK WEIGHING
DEVICE

FIG. 15 DISTRIBUTING TRUCK LOAD-
TRUCK ON LOADOMETER

measurements at the outside of the dual wheel and also at the reference feet of the first Benkelman beam.

Figures 16 and 17 illustrate some of the testing procedures. An extension rod was installed in holes number 4 because of their depth. A wooden plug with a hole in the middle was placed in the top of the pipe to act as a guide for the extension rod. The string shown in Figure 16 was used as an aid in lowering the rod in place and in removing the rod after the tests.

Test Results

Field Data

The data analysed and discussed in this section are those of the beam which was used between the tires of the right dual wheel. The "outside of dual wheel" data and the measurements taken at the reference feet of the beam are discussed briefly.

Limitations of Measurements. In order to convert field data into deflection results, dial readings were multiplied by a factor of 1.855. Figure 18 illustrates the geometry of the Benkelman beam. Dial readings were recorded to the nearest 0.0005 inch. Hence, deflection measurements were recorded to the nearest 0.00093 inch. However, it is doubtful whether the beam reading can be recorded consistently with greater accuracy than the nearest one dial interval or 0.001 inch.

Measurements at Foot of Benkelman Beam. Figure 18 also shows the geometry of the Benkelman beam that demonstrates the effect of beam reference feet movement on the final deflection measurements. If the feet moved 0.001 inch, an error of 0.0027 inch is introduced. A dial

FIG.16 TECHNIQUE OF USING DEEP
HOLE No. 4

FIG.17 BENKELMAN BEAM TESTING
PROCEDURES

FIG.18 GEOMETRY OF THE BENKELMAN BEAM

movement of one interval of the check beam would indicate an error of 0.005 inch in the deflection reading. Conversely, an error of one dial unit in the check beam would introduce an error of 0.005 inch in the deflection reading if it was used as a correction value. Therefore, with an instrument such as the Benkelman beam, where there is an inherent potential instrument error of about one dial unit, it is not possible to check the feet movement of another beam when it can introduce an error of as much as 0.005 inch in the final deflection measurement.

Fortunately, the data obtained in this study indicated that there was no real movement of its reference feet. When the truck was over the probe of the beam, the load was exactly 7.5 feet away from the feet of the beam. If movement had taken place, with the truck in that position movement of the probe would have been recorded. But in only four out of 160 tests did a dial reading of other than zero result when the truck was 7.0 or more feet away from the probe. It must be concluded that on this basis significant movement of the reference feet did not occur.

Outside of Dual Wheel Data. These data were originally intended to be used as controls for the hole data. In this way, any significant difference in the surface reading outside of the dual wheel for each hole could be used to explain unusual hole deflection readings. Although these data indicated relative uniformity of total surface deflection from hole to hole at a single location, they were not used as a precise correction statistic for the hole deflection data because of the greater inherent variability of outside dual wheel deflection measurements.

Layered System Deflection Results

Table 9 presents the deflection results used for the analysis of this phase of the project. These results represent the maximum change in deflection recorded from the previously described loading and unloading cycle of the truck. As an example, if the dial on the beam indicated ~~maximum~~ and minimum readings of .009 and -.001 inches for the entire cycle, the deflection value used would be .010 inches.

Layer Deflections. Deflections of an individual layer in the pavement can easily be computed by finding the difference in deflection of the rods referenced at the interfaces of the layer in question. The layer deflections are presented in Table 10.

Data Omitted From Analysis. Data taken at Stations 347 and 453 were omitted from the analysis. The reference rods of holes number 2 at each of these locations were held up in the pipe and were not moving properly with the interface of the subbase. Station 347 was the only location where the aluminum foil wrapping was omitted from the bottom of the pipe. At the time of installation it was noted that plaster of Paris had seeped under the pipe and had come up around the reference rod for about one inch. This made it impossible to free the reference rod properly at the time of test. At Station 453, despite the fact that the aluminum foil wrap was used, sufficient plaster of Paris came under the pipe to secure the reference rod. Reference rods in all other holes number 2 seemed to function properly.

Data from the 9,250 pound dual wheel load were also excluded from the analysis. Results from this loading series were consistently lower than those indicated by other load increments. However, this load was

Table 9. Layered System Deflection Data

Station	Rear Axle Load, lbs.	Top of Rod Vertical Movement, 0.001 inch Position of Reading, Hole, Reference on				
		Surface	#1 Base	#2 Subbase	#3 Subgrade	#4 5' Deep
347	12,000	9.3	9.3	6.5	3.7	1.9
	18,000	9.3	9.3	5.6	4.6	0.9
	22,500	16.7	16.7	11.1	11.1	4.2
	26,750	19.5	18.6	12.1	13.9	5.6
304	12,000	11.1	9.3	8.3	4.6	4.6
	18,000	6.5	6.5	7.4	6.5	2.8
	22,500	14.8	14.8	14.8	8.3	5.6
	26,750	20.8	20.4	18.6	11.1	9.3
275	12,000	12.1	8.3	7.4	2.8	0
	18,000	12.1	8.3	7.4	5.6	0.9
	22,500	15.7	14.8	13.0	7.4	0.9
	26,750	20.4	20.4	15.7	8.3	1.9
249	12,000	11.1	9.3	8.3	4.6	0
	18,000	10.2	10.2	8.8	7.4	1.9
	22,500	19.5	15.7	14.8	8.3	0
	26,750	19.5	18.6	17.6	13.9	1.9

Table 9 (Cont'd.). Layered System Deflection Data

Station	Rear Axle Load, lbs.	Top of Rod Vertical Movement, 0.001 inch Position of Reading, Hole, Reference on				
		Surface	#1 Base	#2 Subbase	#3 Subgrade	#4 5' Deep
460	12,000	9.3	8.3	6.5	3.7	0.9
	18,000	9.3	7.9	7.4	3.7	0
	22,500	17.6	14.4	13.0	7.0	0.9
	26,750	18.6	16.7	13.9	8.3	0.9
453*	12,000	10.2	10.2	5.6	5.6	4.6
	18,000	5.6	7.4	4.6	3.7	3.2
	22,500	13.9	13.0	8.3	8.3	5.6
	26,750	13.9	15.3	10.2	8.3	5.6
438	12,000	10.2	9.3	6.5	4.6	2.8
	18,000	10.7	7.4	5.1	3.7	0.9
	22,500	12.1	12.1	8.3	5.6	0.9
	26,750	14.8	13.0	10.2	7.4	1.9
426*	12,000	10.2	10.2	10.2	5.6	4.6
	18,000	9.3	11.1	4.2	3.7	3.2
	22,500	14.8	13.9	12.1	5.6	6.5
	26,750	16.7	16.7	14.8	7.4	7.4

* Holes No. 4 were only 27 and 31 inches deep respectively.

Table 10. Layer Deflections, .001 inch

Layer	Station: 347		Subgrade Type: A-6		Crack Frequency: High	
			Dual Wheel Load, lbs.			
	6,000	9,250	11,250	13,390		
Surface	0.0	0.0	0.0	0.6		
Base	2.8	3.7	5.6	6.8		
Subbase	2.8	1.0	0.0	0.0		
Subgrade - 1	1.8	3.7	6.9	8.3		
Subgrade - 2	1.9	0.9	4.2	5.6		

Layer	Station: 304		Subgrade Type: A-4		Crack Frequency: High	
Surface	1.8	0.0	0.0	0.4		
Base	1.0	0.0	0.0	1.8		
Subbase	3.7	0.9	6.5	7.5		
Subgrade - 1	0.0	3.7	2.7	1.8		
Subgrade - 2	4.6	2.8	5.6	9.3		

Layer	Station: 275		Subgrade Type: A-4		Crack Frequency: Low	
Surface	3.8	3.8	0.9	0.0		
Base	0.9	0.9	1.8	4.7		
Subbase	4.6	1.8	5.6	7.4		
Subgrade - 1	2.8	4.7	6.5	6.4		
Subgrade - 2	0.0	0.9	0.9	1.9		

Layer	Station: 249		Subgrade Type: A-6		Crack Frequency: Low	
Surface	1.8	0.0	3.8	0.9		
Base	1.0	1.4	0.9	1.0		
Subbase	3.7	1.4	6.5	3.7		
Subgrade - 1	4.6	5.5	8.3	12.0		
Subgrade - 2	0.0	1.9	0.0	1.9		

Table 10 (Cont'd.). Layer Deflections, .001 inch

Layer	Station: 460		Subgrade Type: A-2		Crack Frequency: High	
			Dual Wheel Load, lbs.			
	6,000	9,250	11,250	13,390		
Surface	1.0	1.4	3.2	1.9		
Base	1.8	0.5	1.4	2.8		
Subbase	2.8	3.7	6.0	5.6		
Subgrade - 1	2.8	3.7	6.1	7.4		
Subgrade - 2	0.9	0.0	0.9	0.9		
Layer	Station: 453		Subgrade Type: A-2		Crack Frequency: Low	
Surface	0.0	0.0	0.9	0.0		
Base	4.6	2.8	4.7	5.1		
Subbase	0.0	0.9	0.0	1.9		
Subgrade - 1	1.0	0.5	2.7	2.7		
Subgrade - 2 (27")	4.6	3.2	5.6	5.6		
Layer	Station: 438		Subgrade Type: A-1		Crack Frequency: Low	
Surface	0.9	3.3	0.0	1.8		
Base	2.8	2.3	3.8	2.8		
Subbase	1.9	1.4	2.7	2.8		
Subgrade - 1	1.8	2.8	4.7	5.5		
Subgrade - 2	2.8	0.9	0.9	1.9		
Layer	Station: 426		Subgrade Type: A-1		Crack Frequency: High	
Surface	0.0	0.0	0.9	0.0		
Base	0.0	6.9	1.8	1.9		
Subbase	4.6	0.5	6.5	7.4		
Subgrade - 1	1.0	0.5	0.0	0.0		
Subgrade - 2 (31")	4.6	3.2	6.5	7.4		

the first series run, and before the other series were run, a heavy weekend rain occurred.

Also, during the first series, some of the rods were being held up in the pipe by the plaster of Paris used to secure the rod. The significance of this was realized before the other series were begun, and it was made certain that with the exceptions of Stations 347 and 453, all rods were free to move with their layer interfaces. Data from holes No. 4 (deep holes) could not be analyzed, because the data were incomplete and quite variable. Most of the variability was due to the difficulties encountered in the installation of these deep holes.

Analysis of Deflection Data. As far as analysis of results is concerned, emphasis is placed on the relative moduli values which are presented in the next section of this report. However, it is interesting to look at the deflection data in the form of accumulative deflections versus depth. Figures 19 through 24 present the data in this form. The data shown are consistent in so far as the shape of the curves is concerned. It should be noted that most of the deflection appears to be in the subbase and subgrade layers of the pavement.

Relative Modulus

Relative moduli values were obtained for the subgrade, subbase, and base courses for six of the eight locations. These are relative values only, since they are based upon Boussinesq stress distribution and upon the deflection measurements made through holes cased with pipe.

It is recognized that in calculating these relative moduli values, application of elastic theory is made to a problem involving the stress

FIG.19 DEFLECTION vs DEPTH, STATION 249

FIG.20 DEFLECTION vs DEPTH , STATION 304

FIG.21 DEFLECTION vs DEPTH, STATION 275

FIG.22 DEFLECTION vs DEPTH , STATION 460

FIG. 23 DEFLECTION vs DEPTH, STATION 426

FIG. 24 DEFLECTION vs DEPTH, STATION 438

displacement responses of an imperfectly elastic material such as the components of the flexible pavement. For this reason, the values are referred to as relative moduli instead of elastic moduli. However, since the deflections dealt with in this study are not of the permanent type, but are recoverable, this application of elastic theory is worthwhile.

Boussinesq Stress Distribution and Elastic Theory

Justification. Use of the Boussinesq stress distribution theory in this analysis appears to be justified on the basis of the following:

1. Relative moduli values, subject to the limitations of the use of elastic theory previously discussed, are adequate, since they are to be used for comparison purposes only.
2. Work by McMahon and Yoder (22) and the Corps of Engineers (23) indicates that actual measured values of stress at different depths produce stress-depth curves shaped like those of Boussinesq.
3. The work by McMahon and Yoder shows that loading 8 and 12 inch steel plates on a 12 inch base course produces stresses at a total depth of 20 inches less than two percent different from the stresses calculated using Boussinesq's theory. At a total depth of 13 inches the stresses measured by McMahon and Yoder were almost identical to those of Boussinesq.
4. In calculating the subbase and base course moduli values, as will be explained in later paragraphs, only stress differences are used.

Considering the magnitude of loads received by the average highway, the use of elastic theory in the analysis of pavement 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). Performance of the subgrades on the WASHO Road Test was elastic-like in behavior in that practically equal and recoverable deflections were produced by several thousand loads following the conditioning period by initial loads (17).

Thus, even though very small immeasurable plastic deformations were no doubt present, deflections recorded in this study were assumed to be of this recoverable type.

Basic Equation Used. The basic equation of this analysis is from the theory of elasticity and assumes a homogeneous, isotropic material.

$$\frac{dS}{dz} = \frac{1}{E} (\sigma_z - \mu\sigma_r - \mu\sigma_t) \quad 1$$

where S = vertical elastic displacement

E = modulus of elasticity of material

z = vertical stress on element illustrated below

r = horizontal radial stresses on the element

t = horizontal tangential stress on the element

μ = Poisson's ratio

z = depth to the element

The vertical stress on this element for a point load at the surface may be found by using the following equation:

$$\sigma_z = \frac{3P}{2\pi z^2} \frac{1}{\left[1 + \left(\frac{r}{z}\right)^2\right]^{5/2}} \quad 2$$

In equation 2, r is the radial distance from point of application to point of stress computation, P is a point load at the surface, and the other symbols are as before.

Love (24) has integrated this equation over a circular area with the resulting equation (when $r = 0$):

$$\sigma_z = p \left[1 - \frac{z^3}{(a^2 + z^2)^{3/2}} \right] \quad 3$$

where σ_z = vertical stress at z depth below centerline of circular area

p = unit load over area

a = radius of area

The radial stress has similarly been found as:

$$\sigma_r = \frac{p}{z} \left[1 + 2\mu - \frac{2(1+\mu)z}{(a^2 + z^2)^{1/2}} + \frac{z^3}{(a^2 + z^2)^{3/2}} \right] \quad 4$$

Using 0.5 for Poisson's ratio, μ , and assuming $\sigma_r = \sigma_t$, and after substituting equations 3 and 4 into equation 1 and integrating from z to ∞ , we get Boussinesq's settlement equation:

$$s = \frac{3pa^2}{2E(a^2 + z^2)^{1/2}} \quad 5$$

$$\text{or } s = \frac{pa}{E} F' \quad 6$$

where $F' = \frac{3}{2} \frac{1}{\left[1 + \left(\frac{z}{a}\right)^2\right]^{1/2}}$ deflection factor

It should be noted that the deflection factor given in equation 6 is applicable for deflections under the centerline of a loaded circular area only. However, influence charts developed by Newmark (25) were used by Foster and Ahlvin (26) to compute values of the deflection factor for any value of depth z and offset r . Calling this deflection factor F , equation 7 is formed.

$$S = \frac{P a}{E} F \quad 7$$

A chart for determining F is given in Figure 33 in Appendix C.

For this thesis, subgrade moduli values were calculated using equation 7 with the F value found for a depth of 20 inches. The subbase moduli values were calculated using as a deflection factor the difference between F values for depths of 13 and 20 inches (the boundaries of the subbase course). In similar fashion the base course moduli values were calculated, the deflection factor being the difference between F values for depths of 5 and 13 inches (the boundaries of the base course).

These can be represented in the following forms:

$$\text{Subgrade: } S = \frac{P a}{E_3} (F_{20 \text{ to } \infty}) \quad 8$$

$$\text{Subbase: } S = \frac{P a}{E_2} (F_{13 \text{ to } \infty} - F_{20 \text{ to } \infty}) \quad 9$$

$$\text{Base Course: } S = \frac{P a}{E_1} (F_{5 \text{ to } \infty} - F_{13 \text{ to } \infty}) \quad 10$$

In the above equations S is the vertical settlement of the layer concerned.

Contact Pressures. For the purpose of calculating the 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. The true imprint is somewhat elliptical, but for tire pressure used

in these tests the imprint can be approximated by a circle (see Figure 25 which shows the tire imprint of a 9,000 pound load on a dual wheel with 80 psi tire pressure).

Modulus Calculations

Equations 8, 9, and 10 were used to solve for E, using measured deflections. The moduli values for each layer were analysed by a two-way analysis of variance. Significance of a factor was determined by variance ratios or "F" tests (21). Tables 11 through 16 summarize the moduli results and the statistical analyses.

Surface course moduli values were not determined from the deflection measurements, because the Benkelman beam was not sensitive enough to record accurate deflections in the order of magnitude of about 0.001 inch. In many cases no deflection was measured within the surface course; this would result in calculated moduli values of infinity.

Details of the calculation of results are presented in Appendix C. The statistical analysis procedures are essentially the same as those discussed previously, which are presented in Appendix B.

Moduli Values Calculated From Offset Deflections

Moduli values were calculated using the deflection data obtained as the truck backed toward the probe end of the beam. Subgrade moduli were calculated for offset distances (distance of truck axle from beam probe) of one foot and also two feet. Subbase moduli were calculated only at one foot offsets, because at two feet the difference between deflection factors ($F_{13 \text{ to } \infty} - F_{20 \text{ to } \infty}$) approached zero or was negative. Base course moduli could not be calculated, because at both the one and two foot offsets most of the base course deflections were equal to zero. The data

FIG. - 25

PRINTS OF A 10" X 20" TIRE, 12 PLY, OF DIFFERENT
INFLATED PRESSURES UNDER 9000 LBS. WHEEL LOAD. (10)

after Winitoy

Table 11. Subgrade Modulus, E_3 , in 1,000 psi

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

* Moduli values for Stations 347 and 453 are included here in order to determine the effect of soil type. Subbase and base course values are not available for these two stations.

Table 12. Results of Two-Way Analysis of Variance of Subgrade Moduli Values

Source of Variance	F	F _{.1}	Significance
Soil Type, A	2.37	2.49	NS
Crack Frequency, B	0.33	3.07	NS
Interaction: A x B	1.48	2.49	NS

Table 13. Subbase Modulus, E_2 , in 1,000 psi

Station	Dual Wheel Load, (pounds)			Crack Frequency
	6,000	11,250	13,390	
249	13.4	12.8	26.8	Low
304	13.4	12.8	13.2	High
275	10.8	14.9	13.4	Low
460	17.7	13.9	17.7	High
426	10.8	12.8	13.4	High
438	26.1	30.8	35.4	Low

Table 14. Results of Two-Way Analysis of Variance of Subbase Moduli Values

Source of Variance	F	F _{.1}	Significance
Wheel Load, C	.70	2.81	NS
Crack Frequency, B	3.76	3.18	.1 level
Interaction, B x C	.43	2.81	NS

Table 15. Base Course Modulus, E_1 , in 1,000 psi

Station	Dual Wheel Load, (pounds)			Crack Frequency
	6,000	11,250	13,390	
249	49.7	101.2	132.0	Low
304	49.7	101.0	73.2	High
275	55.2	50.6	28.1	Low
460	27.6	65.0	47.2	High
426	55.2	50.6	69.4	High
438	17.8	24.0	47.2	Low

Table 16. Results of Two-Way Analysis of Variance of Base Course Moduli Values

Source of Variance	F	F _{.1}	Significance
Wheel Load, C	1.07	2.81	NS
Crack Frequency, B	0.06	3.18	NS
Interaction: B x C	0.14	2.81	NS

from which the moduli values were calculated and the methods of calculation are presented in Appendix C. The results are presented in Tables 17 and 18.

Summary

Figures 26 through 31 summarize the relative moduli values obtained from the total deflection measurements. Figure 32 shows typical modulus values at different offset distances.

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

In general, the results indicated that the base course had higher relative moduli values than the subgrade, and the subgrade somewhat higher than the subbase. Moduli values calculated at the one and two foot offsets were in general agreement with the values calculated from the zero offset data.

It should be emphasized again that the moduli values computed are relative values only. However, their relative values provided a means whereby the characteristics of one layer could be compared with another.

Table 17. Subgrade and Subbase Modulus at One Foot Offset, E_3 and E_2 , in 1,000 psi

Station	Subgrade			Subbase		
	Dual Wheel Load, pounds			Dual Wheel Load, pounds		
	6,000	11,250	13,390	6,000	11,250	13,390
249	35.5	31.2	24.9	18.2	12.6	41.3
304	35.5	23.6	27.9	9.2	*	*
275	35.5	31.2	27.9	18.2	12.6	11.2
460	52.2	37.9	55.8	18.2	8.1	11.2
426	21.6	37.9	31.8	8.7	12.6	45.8
438	26.8	47.2	44.8	18.2	25.2	21.7

* Recorded deflection of zero, no calculation possible.

Table 18. Subgrade Modulus at Two Foot Offset, E_3 , in 1,000 psi

Station	Dual Wheel Load, pounds		
	6,000	11,250	13,390
249	36.0	31.6	31.4
304	76.0	41.6	31.4
275	36.0	31.6	31.4
460	76.0	41.6	39.0
426	18.5	41.6	31.4
438	24.5	41.6	51.6

FIG.26 . RELATIVE MODULUS OF
PAVEMENT LAYERS, STATION 249

FIG.27 RELATIVE MODULUS OF
PAVEMENT LAYERS, STATION 304

FIG.28 RELATIVE MODULUS OF
PAVEMENT LAYERS , STATION 275

**FIG.29 RELATIVE MODULUS OF
PAVEMENT LAYERS, STATION 460**

**FIG.30 RELATIVE MODULUS OF
PAVEMENT LAYERS, STATION 426**

FIG.31 RELATIVE MODULUS OF
PAVEMENT LAYERS, STATION 438

FIG. 32 **RELATIVE MODULUS vs OFFSET
DISTANCE — STATION 275**

PART IV

DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

DISCUSSION

Results of the preceding sections are briefly reviewed in the order they have appeared; significant information is emphasized and in some cases enlarged upon. The information most pertinent to the cracking that has occurred on the Indiana Test Road is combined and discussed. The question of whether or not total deflections are effective in evaluating pavement distress for the road that was studied is answered. Discussion concerning the effectiveness of the evaluation is included.

Cracking, Rutting, and Surface Deflection

Cracking

The most important conclusions obtained from the cracking survey of the Indiana Test Road are:

1. Crack frequency was not statistically related to subgrade soil type.
2. More cracks have occurred in the traffic lane than in the passing lane.
3. Observation has shown that the cracking has progressed since first appearing.

As shown in Table 1 (page 15) pavements built over each of the four subgrade types showed on the average more than 100 linear feet of cracks per station. Since the subgrade soils ranged from high quality granular materials (A-1-a classification) to those of silty clay (A-6 classification), low subgrade support probably was not responsible for the pavement

cracking. If low subgrade support was the cause of the cracking, one would expect a greater number of cracks in pavement built over the silty clay subgrade. Table 1 shows that this was not the case.

More cracks have occurred in the traffic lane than in the passing lane. Since traffic volumes are considerably lighter in the passing lane than in the traffic lane, the number of repetitions of load would seem to be the factor most related to the number of cracks. The cracking 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. Thus it seems likely that as repetitions of load accumulate in the passing lane, this progression will continue.

Other factors, such as pavement thickness, lateral support, and drainage, could also contribute to the difference in crack frequencies. However, these factors would affect both lanes equally.

It should be emphasized that the pavement cracking at present does not affect the serviceability of the pavement from the standpoint of the motorist. Even where wheel track rutting occurs, the surface is as yet not in a condition that would affect the comfort of the driver.

Surface Deflection and Rutting

Analysis of the surface deflection and wheel track rutting data allowed several important conclusions to be drawn concerning the Indiana Test Road. The more important of these are:

1. No significant relationship was found between crack

frequency and either surface deflection or rutting.

2. Both surface deflection and pavement rutting were shown to be influenced by subgrade type.
3. A relationship existed between rutting and surface deflection data; that is, high surface deflections were associated with the larger rutting values and vice-versa.

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 .01 or .05 level. Table 5, page 27, and the accompanying analysis illustrate this point.

Several highway departments are relying on correlation between surface deflection and pavement distress as a tool for evaluating pavement performance (10) (15) (16). It is natural that those charged with the responsibility of evaluating pavements should want to develop a pavement evaluation procedure using a device which can be moved easily and quickly to obtain large numbers of deflection measurements. The data obtained in this study show clearly that the Indiana Test Road is one example of a highway for which such an evaluation procedure cannot be used. The only success, to this writer's knowledge, of correlating total deflection with pavement distress is reported by Huculak (14). In this instance large permanent deformations caused by an extremely weak subgrade were the symptoms of distress.

Actually there is no reason why there should be a relationship between crack frequency and surface deflection. Surface deflection, which

is the sum of the deformations that occur in all the layers of the pavement structure and of the subgrade, is dependent upon many factors. Gross load, tire pressures, moisture content of the subgrade, density of all the pavement components, volume change characteristics of the subgrade layers, and the season of the year are some of the important factors that affect surface deflection measurements. For example, natural variations in subgrade moisture content from one location to another could cause different deflections, regardless of the cracking that had occurred in the pavement surface. Additional discussion on this subject is given below in the paragraphs concerning layered system deflections.

Perhaps not as important, but nevertheless worth discussing, is the relationship found between rutting and total deflection. Wheel track rutting reflects the permanent volume change characteristics 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. Since surface deflections were also found to be affected by subgrade type, a relationship was found between deflection and rutting.

Layered System Deflections

The relative modulus values which were calculated from the layered system deflection measurements indicated three important points:

1. Subgrade and base course modulus values could not be shown to be related to crack frequency.
2. Areas having different crack frequencies had subbase modulus values that were significantly different at the 0.1 level.

3. In general, the base course had higher modulus values than the subgrade, and the subgrade somewhat higher than the subbase.

Before discussion is continued, the limitation of the modulus calculations should be restated here. The flexible pavement under study consists of materials which are not perfectly elastic. However, since the measured deflections were recoverable and not permanent in nature, the application of elastic theory was considered worthwhile.

Since the calculation of relative moduli takes in account decrease in stresses with increase in pavement depth, these modulus values present a truer overall picture of how the pavement structure behaves than the deflection measurements by themselves. The fact that the subbase was found to have lower relative modulus values than the subgrade was surprising. It is true that the subgrade has been subjected to seven years of heavy traffic, and the deflection measurements were taken during the dry season of the year, but the same can be said for the subbase.

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 drainage criterion mentioned above; however, it is known that there was difficulty encountered in compacting the subbase during its construction due to its instability. It was necessary to add fine stone screenings to the top of the subbase layer so that

construction equipment could be moved across it. This indicates the possibility that the subbase material was not of the quality that was expected.

The fact that 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, compared to the complete lack of relationship found 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 contributed to the occurrence of surface cracks. Also, it should be pointed out that the lowest subbase modulus values were obtained at Station 426 where the most severe cracking of the entire road occurred. The pavement at this location was built over A-1-a (granular) type subgrade.

Finding that the subgrade modulus values were not related to crack frequency agrees with the conclusions drawn in the cracking study. Actually, if there were subgrade weaknesses that were related to pavement cracking, they might not appear during the summer or fall and might be evident only during the spring thaw period. This was not the case for the Indiana Test Road, because there were four major soil types, and high crack frequencies occurred over all of them during the same time period. As pointed out earlier in this discussion, since four distinct soil types were present, if the subgrade was a contributing factor it would have shown up in the crack survey.

Ductility and Penetration Tests

At this point some mention should be made concerning penetration and ductility tests made on asphalt extracted from cores removed from the test pavement. The results of these tests, made in accordance with ASTM designation D113-44 (Ductility) and ASTM Designation D5-52 (Penetration), are presented in Table 19. These results are presented to show that the penetrations and ductilities of the asphalt were not sufficiently low to cause pavement cracking by themselves.

Table 19. Average Ductility, cms, and Penetration, .1mm, Core Layers

Station	Crack Frequency	Surface		Core Component Binder		Base	
		Pene.	Duct.	Pene.	Duct.	Pene.	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

It should be noted that the penetration and ductility values are the lowest for the surface course and the largest for the base course. This undoubtedly is a result of the surface being more oxidized than the base since it was exposed to greater weathering influences. Both the penetration and ductility values are rather low, particularly for the surface.

This could present conditions conducive to pavement cracking. However, these values are not uncommonly low, because it is possible for a pavement to have even lower penetrations and ductilities and not show any distress. Brown, Sparks, and Larsen (28) presented penetration and ductility results from two rigid pavement overlays of asphaltic concrete over a period of nine years. Four different asphalts were used, with original penetrations of 60, 62, 49, and 53 and ductilities of 150~~4~~, 150~~4~~, 49, and 53. Within five years the penetrations were 36, 21, 37, and 24 respectively, and the ductilities were 74, 8, 6, and 6 respectively. At the end of ten years, the pavements were still in perfect condition. Although these surfaces were built over a rigid base, the data support the contention that it was not a brittle nature of the Indiana Test Road surface which was the primary cause of pavement cracking.

Evaluation of Test Pavement

On the basis of the findings from the cracking survey, surface deflection and rutting studies, and the layered system deflection analysis, it is believed that greater than normal deflection in the subbase is an important factor in the cause of cracking that is occurring in the test pavement. The two most important reasons for drawing this conclusion are:

1. Areas of high crack frequency were associated with low values of subbase modulus and vice-versa (see Table 13, page 74).
2. Crack frequency was not found to be associated with type of subgrade nor with the relative modulus values of either the subgrade or base.

The most important supporting fact is that the cracking occurred regardless of the subgrade type over which the pavement was built. If, for example, the subbase was omitted in the design and cracking resulted because of subgrade characteristics, the cracking would undoubtedly be more severe over one subgrade soil type than another. However, with the subbase included in the design and if the cracking was to be attributed to the subbase, because of its relatively uniform nature, one would expect the cracking to occur in all sections of the road.

Cracking in a pavement surface can be caused by many factors including 1) shear stresses, 2) tensile stresses induced by temperature and moisture changes in the surface itself, 3) tensile stresses due to bending caused by deflection of the pavement structure, and 4) stresses due to repeated loads which are bending stresses 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 8, page 35). The cracks that did occur were quite smooth and lacked the evidence of a shear type of failure.

Tensile stresses induced by moisture and temperature changes would be evidenced by the formation of transverse and longitudinal cracks in somewhat regular patterns, similar to those commonly found in portland cement concrete. This is probably not the situation at the test road,

although it could be a secondary factor. If the ductility and penetration results were considerably lower than reported, this cause would be more likely.

Failure due to tensile stresses resisting bending may or may not result in permanent deformation, but only a limited number of loads would be required to cause the failure. Since considerably fewer cracks have occurred in the passing lane than in the traffic lane, and since essentially equivalent conditions exist under the pavement surface of each lane, number of load repetitions must be an important factor.

In conclusion, repeated loadings leading to excessive deformation in the subbase is considered to be the major factor causing the pavement cracking. It should be noted again that measurement of surface deflections alone were inadequate in drawing this conclusion.

Effectiveness of Evaluation Methods

The importance of being able to evaluate causes of various pavement defects has been discussed earlier in this thesis and should be emphasized again at this point. Flexible pavement design methods cannot be evaluated properly unless the reasons for their shortcomings can be made known. In order to do this adequately, each component of the pavement structure should be examined.

The procedures used in this study provided a basis upon which to develop reasonable answers to the question of what was causing the cracking of the test pavement. However, the problem is of such complexity as to make impossible complete explanations of all of the 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. Most of the initial cracks were essentially longitudinal with some of the later 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. One such answer would point out that the cracks, although mostly longitudinal, are quite jagged and irregular. Like breaking a large piece of glass, one crack encourages additional cracking, and with the traffic moving in only one direction, a general longitudinal direction could be encouraged. Also, since 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

It is important that one of the specific test methods used be discussed here. The method of making the surface deflection measurements differs from that developed by the WASHO Test Road personnel (7). In the WASHO method the probe of the Benkelman beam is placed between the dual wheels to a distance of 4.5 feet under the truck. The truck then pulls forward and initial and maximum dial readings are recorded. This

puts the dual wheels initially about three feet 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 feet is recommended over that developed at the WASHO Road Test.

SUMMARY OF CONCLUSIONS AND RESULTS

Concerning the pavement cracking of the Indiana Test Road sections, the following conclusions are drawn:

1. Total deflections, by themselves, were not effective in determining the cause of the distress.
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 a particular 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 three feet from the reference feet of the beam.

RECOMMENDATIONS

Recommendations in three areas are made. They deal with the procedures of the layered system study, the location of the deflection holes, and the obtaining of springtime deflection measurements.

Procedures of Layered System Study

Installation of Holes

The need for several modifications in the installation procedure in future work is indicated. One of these modifications concerns the use of plaster of Paris for surrounding the pipe casing. Plaster of Paris should still be used, but if a fine sand was poured in initially to fill the bottom inch or two of the void space, a barrier could be made to prevent the liquid plaster of Paris from seeping under the pipe and interfering with the action of the reference rod. This occurred on several occasions resulting in the need for discarding data from two holes. It is believed that the plaster of Paris was quite efficient in filling the voids surrounding the pipe casing. Also, a field test indicated that plaster of Paris did not bond very well to the oiled pipe.

A second modification should be that no attempt should be made to secure the reference rod to the bottom of the hole. If a firm, solid base was made with the plaster of Paris, and care taken that it did not reach the sides of the casing, the reference rod could then be placed satisfactorily each time a deflection measurement was desired.

For casing the deep holes (number four) for the entire depth, the drilling procedures were not entirely satisfactory. Casing was used for the entire depth at two locations, but great difficulty was experienced in doing so. In these cases, the pipe had to be driven the last two feet or so because it was not possible to drill a straight hole to that depth. When the subgrade was cohesive, leaving the hole uncased for the final three feet proved satisfactory.

Since the deep holes did not produce consistent data, it would perhaps be better to use a lesser depth, say at about two feet below the top of the subgrade. This would be shallow enough to record definite deflections, and deep enough to define the strength of the subgrade.

The hole caps seemed to perform very satisfactorily. In fact, upon several occasions when the lunch hour interrupted the test series, the caps were placed in the holes without the nails, and traffic was allowed over them. In no case was a cap lost. At this writing, three months after the tests, only two caps have been lost, apparently due to a defective weld of the top plate to the tube which inserts into the pipe.

Test Procedures

No major changes are recommended in the test procedures. The procedures proved to be efficient, and results were produced of sufficient accuracy to fulfill the purpose of the project.

The procedure of backing the truck toward the probe point from a distance of ten feet away and then having the truck immediately move forward worked satisfactorily. In this manner, the applied load was transient to a degree, yet it remained over the point long enough for a maximum deflection value to be obtained. However, this procedure would have been

of greater value if a more precise method was available for indicating when the rear axle of the truck was exactly opposite the individual foot marks. This would have made the data adaptable for radius of curvature measurements.

A highly competent driver operated the trucks. Without him the work would have been extremely difficult to perform. A good truck driver is an essential part of undertaking such a project.

On the basis of previous discussion, the foot measurements and the side of dual wheel measurements are not recommended. Instead, if a second beam was available, surface measurements should be taken between the tires of the left rear dual wheel as well as between the tires of the right wheel. This would give a relatively valid deflection value with which to correct the hole measurements for hole to hole differences of total deflection.

Traffic Lane Deflection Holes

In the case of the Indiana Test Road, there was considerably more cracking in the traffic lane than in the passing lane. If deflection holes were installed in the traffic lane instead of the passing lane, a greater spread between low and high crack frequency for each of the subgrade types could have been obtained. This might have resulted in better correlation between crack frequency and the causative factors. The deflection holes stood up well enough under the passing lane traffic to make it seem likely that they would also hold up under the traffic of the traffic lane.

Springtime Measurements

In many cases, springtime measurements would be invaluable. Subgrade weaknesses that were related to pavement cracking might not appear during the summer or fall and would only be evident during the spring thaw period. This was not the case for the Indiana Test Road, because there were four major subgrade soil types and high crack frequencies occurred over all of them. However, if a highway was tested that had only similar subgrade soil types, tests run when the subgrade was at its weakest would be the only way of truly evaluating the role of the subgrade. In the case of the Indiana Test Road, even though the subgrade was eliminated early as a causative factor of the pavement cracking, it would be valuable to know how the relative strength of the subgrade compares to the subbase and base courses during the spring thaw period. Little is known about the behavior of the different layers of a pavement during the critical period of the year, so that any additional information concerning the spring thaw period would be an important contribution to the small amount of information already available.

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APPENDIX A

Tables Relating 1959 and 1960 Total Deflection
Data to Subgrade Soil Type, Crack Frequency,
and Pavement Surface Thickness

TABLE 20

Relationship Between Soil Type and Total Deflection
1959 Data

Soil Type	Recording Position	Total Deflection, 0.001 inch				
		1	2	3	4	5
A-1	Avg. Tot. Deflection	9	14	6	14	8
	Maximum Deflection	17	20	10	20	13
	Minimum Deflection	2	10	1	11	5
	Range	15	10	9	9	8
A-2	Avg. Tot. Deflection	10	18	7	18	10
	Maximum Deflection	19	26	18	21	12
	Minimum Deflection	0	12	1	15	6
	Range	19	14	17	6	6
A-4	Avg. Tot. Deflection	15	25	8	22	12
	Maximum Deflection	21	36	14	26	13
	Minimum Deflection	7	17	4	20	8
	Range	14	19	10	6	5
	Avg. Tot. Deflection	14	26	6	22	13
	Maximum Deflection	22	33	9	34	19
	Minimum Deflection	6	17	2	17	9
	Range	16	16	7	17	10

TABLE 21

Relationship Between Soil Type and Total Deflection
1960 Data

Soil Type	Recording Position	Total Deflection, 0.001 inch				
		1	2	3	4	5
A-1	Avg. Max. Deflection	9	15	3	14	9
	Maximum Deflection	16	21	5	20	11
	Minimum Deflection	6	11	1	11	5
	Range	10	10	4	9	6
A-2	Avg. Max. Deflection	10	15	3	13	8
	Maximum Deflection	15	20	5	17	9
	Minimum Deflection	4	9	2	10	7
	Range	11	11	3	7	2
A-4	Avg. Max. Deflection	12	21	4	18	11
	Maximum Deflection	18	30	6	25	14
	Minimum Deflection	7	10	2	10	3
	Range	11	20	4	15	11
A-6	Avg. Max. Deflection	13	20	4	18	11
	Maximum Deflection	27	34	6	34	17
	Minimum Deflection	4	13	2	12	8
	Range	23	21	4	22	9

TABLE 22

Relationship Between Soil Type and Total Deflection
Combined 1959 and 1960 Data

Soil Type	Recording Position	Total Deflection, 0.001 inch				
		1	2	3	4	5
A-1	Avg. Max. Deflection	9	14	5	14	9
	Maximum Deflection	16	21	8	20	13
	Minimum Deflection	2	10	1	11	5
	Range	14	11	7	9	8
A-2	Avg. Max. Deflection	10	16	4	15	9
	Maximum Deflection	19	26	8	22	12
	Minimum Deflection	1	9	2	8	6
	Range	18	17	6	14	6
A-4	Avg. Max. Deflection	14	23	5	21	12
	Maximum Deflection	24	36	14	26	14
	Minimum Deflection	7	10	2	10	3
	Range	17	26	12	16	11
A-6	Avg. Max. Deflection	14	23	5	20	11
	Maximum Deflection	27	34	12	34	17
	Minimum Deflection	4	13	2	12	8
	Range	23	21	10	22	9

TABLE 23

Relationship Between Lanes and Deflections
1959 Data

Soil Type	Recording Pt. Lane	Total Deflection, 0.001 inch									
		1		2		3		4		5	
		Tra.Pas.		Tra.Pas.		Tra.Pas.		Tra.Pas.		Tra.Pas.	
A-1	Avg. Tot. Defl.	9	8	16	13	8	4	16	13	11	6
	Maximum	14	17	20	14	10	8	20	14	13	7
	Minimum	6	2	10	11	7	1	12	11	10	5
	Range	8	15	10	3	3	7	8	3	3	2
A-2	Avg. Tot. Defl.	13	7	20	16	9	6	18	18	10	10
	Maximum	19	10	26	19	18	7	21	20	11	12
	Minimum	6	1	17	12	1	4	15	15	6	9
	Range	13	9	9	7	17	3	6	5	5	3
A-4	Avg. Tot. Defl.	18	12	28	23	8	7	23	22	13	11
	Maximum	25	19	32	36	14	10	25	26	14	13
	Minimum	12	7	23	16	6	4	20	18	11	9
	Range	13	12	9	20	8	6	5	8	3	4
A-6	Avg. Tot. Defl.	17	11	29	23	6	7	24	20	14	13
	Maximum	22	16	33	30	9	9	34	23	19	20
	Minimum	12	6	21	17	2	6	19	17	11	9
	Range	10	10	12	13	7	3	15	6	8	11
Over-all Average		14	9	23	19	8	6	20	18	12	10

TABLE 24

Relationship Between Lanes and Deflections
1960 Data

Soil Type	Recording Pt. Lane	Total Deflection, 0.001 inch									
		1	2	3	4	5					
		Tra.Pas.	Tra.Pas.	Tra.Pas.	Tra.Pas.	Tra.Pas.					
A-1	Avg. Tot. Defl.	11	7	15	14	4	3	16	13	10	8
	Maximum	16	8	21	16	5	4	20	15	11	9
	Minimum	7	6	11	12	3	1	13	11	9	5
	Range	9	2	10	4	2	3	7	4	2	4
A-2	Avg. Tot. Defl.	13	7	18	12	4	3	15	11	8	9
	Maximum	15	9	20	16	5	3	17	13	9	9
	Minimum	10	4	17	9	3	2	12	10	7	7
	Range	5	5	3	7	2	1	5	3	2	2
A-4	Avg. Tot. Defl.	14	10	25	17	5	3	22	15	11	11
	Maximum	18	14	30	24	6	5	25	19	19	14
	Minimum	10	7	20	10	4	2	18	10	3	9
	Range	8	7	10	14	2	3	7	9	16	5
A-6	Avg. Tot. Defl.	19	8	25	15	5	3	22	15	11	10
	Maximum	27	10	34	17	6	4	34	17	17	12
	Minimum	16	4	18	13	3	2	14	12	8	9
	Range	11	6	16	4	3	2	20	5	9	3
Over-all Average		14	8	21	14	4	3	18	14	10	10

TABLE 25

Relationship Between Lanes and Deflections
Combined 1959 and 1960

Soil Type	Recording Pt. Lane	Total Deflection, 0.001 inch									
		1		2		3		4		5	
		Tra.Pas.		Tra.Pas.		Tra.Pas.		Tra.Pas.		Tra.Pas.	
A-1	Avg. Tot. Defl.	10	7	16	13	6	4	16	13	10	6
	Maximum	16	16	21	16	10	8	20	15	13	9
	Minimum	6	2	10	11	3	1	12	11	9	5
	Range	10	14	11	5	7	7	8	4	4	4
A-2	Avg. Tot. Defl.	13	7	19	14	4	4	17	13	9	9
	Maximum	19	10	26	19	8	7	22	20	12	12
	Minimum	6	1	17	9	2	2	12	8	6	7
	Range	13	9	9	10	6	5	10	12	6	5
A-4	Avg. Tot. Defl.	16	11	27	20	7	5	22	19	12	12
	Maximum	24	19	32	36	14	10	25	26	19	14
	Minimum	10	7	20	10	4	2	18	10	3	9
	Range	14	12	12	26	10	8	7	16	16	5
A-6	Avg. Tot. Defl.	18	10	27	19	5	5	23	17	12	11
	Maximum	27	16	34	30	8	9	34	23	17	14
	Minimum	12	4	18	13	2	2	14	12	8	9
	Range	15	12	16	17	6	7	20	11	9	5
Over-all Average		14	9	22	16	5	5	19	16	11	9

TABLE 26

Relationship Between Crack Frequency and Deflection
1959 Data

Soil Type	Recording Pt. Crack Freq.	Total Deflection, 0.001 inch									
		1		2		3		4		5	
		Low	High	Low	High	Low	High	Low	High	Low	High
A-1	Avg. Tot. Defl.	12	6	16	12	7	5	16	13	8	8
	Maximum	17	8	20	15	10	7	20	15	12	13
	Minimum	7	2	13	10	4	1	12	11	5	5
	Range	10	6	7	5	6	6	8	4	7	8
A-2	Avg. Tot. Defl.	11	9	18	17	5	9	17	19	9	11
	Maximum	14	19	19	26	7	18	20	21	12	11
	Minimum	7	1	17	12	1	4	15	17	6	10
	Range	7	18	2	14	6	14	5	4	6	1
A-4	Avg. Tot. Defl.	17	14	29	22	7	8	22	23	11	13
	Maximum	25	21	36	26	14	10	26	25	12	14
	Minimum	7	9	17	16	4	7	18	21	8	12
	Range	18	12	19	10	10	3	8	4	4	2
A-6	Avg. Tot. Defl.	15	13	24	27	7	6	21	23	12	12
	Maximum	17	22	32	33	9	9	25	34	13	19
	Minimum	12	6	21	17	6	2	18	17	9	10
	Range	5	16	11	16	3	7	7	17	4	9
Over-all Average		13	10	22	20	7	7	19	19	10	11

TABLE 27

Relationship Between Crack Frequency and Deflection
1960 Data

Soil Type	Recording Pt. Crack Freq.	Total Deflection, 0.001 inch									
		1		2		3		4		5	
		Low	High	Low	High	Low	High	Low	High	Low	High
A-1	Avg. Tot. Defl.	10	8	16	13	3	3	15	14	10	9
	Maximum	16	9	21	16	5	4	20	15	11	10
	Minimum	6	7	13	11	1	3	11	13	9	5
	Range	10	2	8	5	4	1	9	2	2	5
A-2	Avg. Tot. Defl.	10	9	16	15	3	3	12	14	8	8
	Maximum	15	14	20	18	3	5	16	17	9	9
	Minimum	6	4	9	11	3	2	10	10	7	7
	Range	9	10	11	7	0	3	6	7	2	2
A-4	Avg. Tot. Defl.	11	13	22	20	4	4	19	18	12	11
	Maximum	14	18	30	26	6	5	25	24	19	12
	Minimum	8	7	13	10	2	2	14	10	3	9
	Range	6	11	17	16	4	3	11	14	16	3
A-6	Avg. Tot. Defl.	13	14	19	20	4	4	17	20	10	12
	Maximum	16	27	24	34	6	6	20	34	11	17
	Minimum	8	4	14	13	2	3	12	14	9	8
	Range	8	23	10	21	4	3	8	20	2	9
Over-all Average		11	11	18	17	4	4	16	16	10	10

TABLE 28

Relationship Between Crack Frequency and Deflection
Combined 1959 and 1960

Soil Type	Recording Pt. Crack Freq.	Total Deflection, 0.001 inch									
		1		2		3		4		5	
		Low	High	Low	High	Low	High	Low	High	Low	High
A-1	Avg. Tot. Defl.	11	7	16	13	5	4	15	14	9	9
	Maximum	16	9	21	16	10	7	20	15	12	13
	Minimum	6	2	13	10	1	1	11	12	5	5
	Range	10	7	8	6	9	6	9	3	7	8
A-2	Avg. Tot. Defl.	10	9	17	16	4	9	15	15	9	9
	Maximum	15	19	20	26	7	8	20	22	12	12
	Minimum	6	1	9	11	2	2	10	8	6	7
	Range	9	18	11	15	5	6	10	14	6	5
A-4	Avg. Tot. Defl.	14	14	25	21	6	6	21	21	12	12
	Maximum	24	21	36	26	14	10	26	25	19	14
	Minimum	7	7	13	10	2	2	14	10	3	9
	Range	17	14	23	16	12	8	12	15	16	5
A-6	Avg. Tot. Defl.	14	14	22	24	6	5	19	22	11	12
	Maximum	17	27	32	34	9	8	25	34	14	17
	Minimum	8	4	14	13	2	2	16	14	9	8
	Range	9	23	18	21	7	6	9	20	5	9
Over-all Average		12	11	20	18	5	6	17	18	11	10

TABLE 29

Relationship Between Surface Thickness and Deflection
1959 Data

Soil Type	Recording Pt. Surf. Thickn.	Total Deflection, 0.001 inch									
		1		2		3		4		5	
		5+	5-	5+	5-	5+	5-	5+	5-	5+	5-
A-1	Avg. Tot. Defl.	8	9	14	15	6	7	13	15	8	9
	Maximum	17	14	18	20	8	10	16	20	10	12
	Minimum	2	6	10	11	1	4	11	12	5	5
	Range	15	8	8	9	7	6	5	8	5	7
A-2	Avg. Tot. Defl.	11	9	18	17	6	8	17	19	9	11
	Maximum	19	12	26	19	8	18	21	20	11	12
	Minimum	1	6	12	13	4	1	15	17	6	10
	Range	18	6	14	6	4	7	6	3	5	2
A-4	Avg. Tot. Defl.	15	13	25	28	9	7	21	24	12	13
	Maximum	25	19	32	36	14	10	23	26	14	14
	Minimum	7	11	16	23	5	4	18	21	8	12
	Range	18	8	26	13	9	6	4	5	6	2
A-6	Avg. Tot. Defl.	11	17	28	23	6	7	20	25	11	15
	Maximum	16	22	32	33	9	9	25	34	14	20
	Minimum	6	12	22	17	2	6	18	19	9	11
	Range	10	10	10	16	7	3	7	15	5	9
Over-all Average		11	12	21	21	7	7	18	21	11	12

TABLE 30

Relationship Between Surface Thickness and Deflection
1960 Data

Soil Type	Recording Pt. Surf. Thickn.	Total Deflection, 0.001 inch									
		¹		²		³		⁴		⁵	
		5+	5-	5+	5-	5+	5-	5+	5-	5+	5-
A-1	Avg. Tot. Defl.	8	10	13	16	3	3	13	15	8	10
	Maximum	11	16	15	21	4	5	15	20	10	11
	Minimum	6	7	11	14	3	1	11	13	5	9
	Range	5	9	4	7	1	4	4	7	5	2
A-2	Avg. Tot. Defl.	10	10	14	17	3	3	13	13	9	8
	Maximum	14	15	17	20	5	3	15	17	9	9
	Minimum	4	6	9	12	2	3	11	10	7	7
	Range	10	9	8	8	3	0	4	7	2	2
A-4	Avg. Tot. Defl.	11	14	17	25	4	5	16	22	8	14
	Maximum	18	16	26	30	6	5	20	25	10	19
	Minimum	7	12	10	20	2	4	10	18	3	12
	Range	11	4	16	10	4	1	10	7	7	7
A-6	Avg. Tot. Defl.	12	15	17	22	4	4	16	21	9	13
	Maximum	16	27	24	34	6	6	20	34	10	17
	Minimum	4	8	13	15	2	3	12	14	8	10
	Range	12	19	11	19	4	3	8	20	2	7
Over-all Average		10	12	15	20	3	4	14	18	8	11

TABLE 31

Relationship Between Surface Thickness and Deflection
Combined 1959 and 1960 Data

Soil Type	Recording Pt. Surf. Thickn.	Total Deflection, 0.001 inch									
		1		2		3		4		5	
		5+	5-	5+	5-	5+	5-	5+	5-	5+	5-
A-1	Avg. Tot. Defl.	8	9	13	16	5	5	13	15	8	9
	Maximum	16	16	18	21	8	10	16	20	10	13
	Minimum	2	6	10	11	1	1	11	12	5	5
	Range	14	10	8	10	7	9	5	8	5	8
A-2	Avg. Tot. Defl.	10	9	16	17	5	4	15	15	9	9
	Maximum	19	15	26	20	8	7	22	20	12	12
	Minimum	1	6	9	12	2	2	11	8	6	7
	Range	18	9	17	8	6	5	11	12	6	5
A-4	Avg. Tot. Defl.	13	15	20	26	6	6	18	23	10	14
	Maximum	24	19	32	36	14	10	23	26	14	19
	Minimum	7	12	10	20	2	4	10	18	3	12
	Range	17	7	22	16	12	6	13	8	11	7
A-6	Avg. Tot. Defl.	11	16	23	23	5	6	18	23	10	13
	Maximum	16	27	32	34	9	8	25	34	14	17
	Minimum	4	8	13	15	2	3	12	14	8	10
	Range	12	19	19	19	7	5	13	20	6	7
Over-all Average		11	12	18	20	5	5	16	19	9	11

APPENDIX B

Statistical Model Example for Analysis of Variance

Statistical Model Example for Analysis of Variance

The statistical model presented here is for the analysis of variance of the 1959 and 1960 total deflection values. Models for all the other analyses presented in this thesis would be identical except for the name and number of factors.

Model:

$$\begin{aligned}
 Y_{ijklm} = & \mu + \alpha_i + \beta_j + (\alpha\beta)_{ij} + \gamma_k + (\alpha\gamma)_{ik} \\
 & + (\beta\gamma)_{jk} + (\alpha\beta\gamma)_{ijk} + \delta_l + (\alpha\delta)_{il} \\
 & + (\beta\delta)_{jl} + (\gamma\delta)_{kl} + (\beta\gamma\delta)_{jkl} \\
 & + (\alpha\gamma\delta)_{ikl} + (\alpha\beta\delta)_{ijl} + (\alpha\beta\gamma\delta)_{ijkl} \\
 & + \epsilon_{m(ijkl)}
 \end{aligned}$$

μ , constant

α , soil type, $i = 1$ to 4, fixed

β , lane, $j = 1$ to 2, fixed

γ , crack frequency level, $k = 1$ to 2, fixed

δ , surface thickness level, $l = 1$ to 2, fixed

ϵ , variable, $m = 1$ to 4, fixed

To find the mean squares of each factor, procedures presented in reference 27 were used. The error mean square of $\epsilon_{m(ijkl)}$ was used to test for significance the mean squares of all the other factors.

Complete results of the mean square calculations and the subsequent F tests for significance are shown in Table 32.

The assumption is made that the standard deviations of the data within a certain factor are similar enough to be homogeneous. This is a valid assumption since the variability of all deflection measurements of a particular type to be analysed (total, subgrade, subbase, or base)

should not be affected by crack frequency, subgrade type, surface thickness, or wheel load. If the data were analysed so as to compare total versus subbase or other similar comparisons, the assumption may not be valid.

TABLE 32

Mean Square Calculations and F Tests For Total Deflection Data

Source of Variance	Sums of Squares	Degrees of Freedom	Mean Squares	F	F .05	F .01	Significance
A, Soil Type (α)	1312	3	437	36.4	2.71	4.03	.01 level
B, Lane (β)	612	1	612	51.0	3.95	6.96	.01 level
C, Crack Frequency Level (γ)	24	1	24	2.0	3.95	6.96	NS
D, Surface Thickness Level (δ)	200	1	200	16.7	3.95	6.96	.01 level
Interactions: A x B	84	3	28	2.3	2.71	4.03	NS
A x C	105	3	35	2.9	2.71	4.03	.05 level
B x C	16	1	16	1.3	3.95	6.96	NS
A x B x C	78	3	26	2.2	2.71	4.03	NS
A x D	134	3	45	3.8	2.71	4.03	.05 level
B x D	13	1	13	1.1	3.95	6.96	NS
C x D	0	1	0	0	3.95	6.96	NS
B x C x D	60	1	60	5.0	3.95	6.96	.05 level
A x B x D	175	3	58	4.8	2.71	4.03	.01 level
A x C x D	252	3	84	7.0	2.71	4.03	.01 level
A x B x C x D	294	3	98	7.9	2.71	4.03	.01 level
Error, (ϵ)	1170	96	12	-	-	-	-
Total	4529	127	-	-	-	-	-

APPENDIX C

Supplemental Stress, Deflection Factor,
and Modulus Calculations

VERTICAL DEFLECTION
CIRCULAR LOAD
POISSON'S RATIO = 0.5

FIG. 33

FIG. 34

001-530

TABLE 33

Centerline Stress and Deflection Factors Under 6000 lb. Dual Wheel

Depth (inches)	Depth (radii)	% Stress One Wheel	% Stress Two Wheels	Q _L Stress (psi)	Deflection Factor, F	
					F One Wheel	F Q _L
0	0	0	0	0	.36	.72
2.5	.73	2.3	4.6	3.7	-	-
5.0	1.45	6.3	12.6	10.1	.40	.80
9.0	2.60	6.7	13.4	10.7	-	-
13.0	3.77	5.5	11.0	8.8	.31	.62
16.5	4.80	4.2	8.4	6.7	-	-
20.0	5.80	3.1	6.2	5.0	.22	.44
23.5	6.80	2.3	4.6	3.7	-	-
25.5	7.40	2.1	4.2	3.4	-	-
27.0	7.80	2.0	4.0	3.2	.17	.34
31.0	9.00	1.5	3.0	2.4	.16	.32

Dual Wheel Load: 6000 lbs.

Contact Area, each tire = 37.5 in.²

Radius, equivalent circular tire imprint = 3.45 in.

Offset, to Q_L, in radii = 6.75/3.45 = 1.96 2

Contact Pressure: 80 psi

TABLE 34

Centerline Stress and Deflection Factors Under 9,250 lb. Dual Wheel

Depth (inches)	Depth (radii)	% Stress One Wheel	% Stress Two Wheels	Q _L Stress (psi)	Deflection Factor, F	
					F One Wheel	F Q _L
0	0	0	0	0	.52	1.04
2.5	.58	6.5	13.0	10.4	-	-
5.0	1.17	12.0	24.0	19.2	.53	1.06
9.0	2.10	11.6	23.2	18.6	-	-
13.0	3.04	8.5	17.0	13.6	.36	.72
16.5	3.85	6.2	12.4	9.9	-	-
20.0	4.67	5.0	10.0	8.0	.26	.52
23.5	5.48	3.8	7.6	6.1	-	-
25.5	5.96	3.2	6.4	5.1	-	-
27.0	6.30	3.0	6.0	4.8	.21	.42
31.0	7.24	2.2	4.4	3.5	.18	.36

Dual Wheel Load: 9,250 lbs.

Contact Area, each tire = 57.8 in.²

Radius, equivalent circular tire imprint = 4.29 inches

Offset, to Q_L, in radii = 6.75/4.29 = 1.57 1.5

Contact Pressure: 80 psi

TABLE 35

Centerline Stress and Deflection Factors Under 11,250 lb. Dual Wheel

Depth (inches)	Depth (radii)	% Stress One Wheel	% Stress Two Wheels	C _L Stress (psi)	Deflection Factor, F	
					F One Wheel	F C _L
0	0	0	0	0	.52	1.04
2.5	.53	6.5	13.0	10.4	-	-
5.0	1.05	11.8	23.6	18.9	.54	1.08
9.0	1.90	12.1	24.2	19.3	-	-
13.0	2.75	10.0	20.0	16.0	.41	.82
16.5	3.49	7.6	15.2	12.1	-	-
20.0	4.22	5.5	11.0	8.8	.30	.60
23.5	4.96	4.3	8.6	6.9	-	-
25.5	5.38	4.0	8.0	6.4	-	-
27.0	5.70	3.4	6.8	5.4	.23	.46
31.0	6.54	2.8	5.6	4.5	.21	.42

Dual Wheel Load: 11,250 lbs.

Contact Area, each tire = 70.3 in.²

Radius, equivalent circular tire imprint = 4.74 inches

Offset, to C_L, in radii, = $6.75/4.74 = 1.43$ 1.5

Contact Pressure: 80 psi

TABLE 36

Centerline Stress and Deflection Factors Under 13,390 lb. Dual Wheel

Depth (inches)	Depth (radii)	% Stress One Wheel	% Stress Two Wheels	CL Stress (psi)	Deflection Factor, F	
					F One Wheel	F CL
0	0	0	0	0	.63	1.26
2.5	.48	13.2	26.4	21.1	-	-
5.0	.97	20.0	40.0	31.9	.62	1.24
9.0	1.74	16.3	32.6	24.5	-	-
13.0	2.52	11.6	23.2	18.5	.44	.88
16.5	3.19	9.1	18.2	14.5	-	-
20.0	3.87	7.0	14.0	11.2	.32	.64
23.5	4.16	6.3	12.6	10.1	-	-
25.5	4.94	4.6	9.2	7.4	-	-
27.0	5.24	4.4	8.8	7.0	.26	.52
31.0	6.01	3.2	6.4	5.1	.23	.46

Dual Wheel Load: 13,390 lbs.

Contact Area, each tire = 83.5 in.²

Radius, equivalent circular tire imprint = 5.16 inches

Offset, to CL, in radii = $6.75/5.16 = 1.31$ 1.25

Contact Pressure: 80 psi

Elastic Modulus Calculation Procedures

Subgrade Modulus

Table 37 summarizes the data required for the modulus calculations.

TABLE 37

Subgrade Modulus Calculation Data, Depth 20 Inches

Dual Wheel Load, lbs.	Deflection Factor, $F_{20 \text{ to } \infty}$	Tire Imprint Radius, a, inches	Contact Pressure psi
6,000	.44	3.45	80
11,250	.60	4.74	80
13,390	.64	5.16	80

Using equation 8, and solving for subgrade modulus, E_3 , then

$$E_3 = \frac{P}{S} a F_{20 \text{ to } \infty} \quad 11$$

where S is the recorded deflection of hole number 3 (interface of subgrade and subbase). Substituting the values from Table 41 in equation 11, the following equations for subgrade modulus are obtained:

$$6,000 \text{ lb. wheel load, } E_3 = \frac{80 \times 3.45 \times .44 \text{ lb./in.}}{S} = \frac{121.4}{S} \quad 12$$

$$11,250 \text{ lb. wheel load, } E_3 = \frac{80 \times 4.74 \times .60 \text{ lb./in.}}{S} = \frac{227.5}{S} \quad 13$$

$$13,390 \text{ lb. wheel load, } E_3 = \frac{80 \times 5.16 \times .64 \text{ lb./in.}}{S} = \frac{264.5}{S} \quad 14$$

Subgrade modulus can then be obtained by substituting deflection values obtained at hole 3 (Table 9) in equations 12, 13, and 14.

Subbase Modulus

Table 38 summarizes the data required for the modulus calculations.

Using equation 9, and solving for subbase modulus, E_2 , then

$$E_2 = \frac{P a}{S} (F_{13 \text{ to } \infty} - F_{20 \text{ to } \infty}) \quad 15$$

where S is the recorded deflection of the subbase (deflection at hole number 2 minus that recorded at hole number 3). Substituting values from Table 38 in equation 15, the following equations for subgrade modulus are obtained:

$$6,000 \text{ lb. wheel load, } E_2 = \frac{80 \times 3.45 \times .18 \text{ lb./in.}}{S} = \frac{49.7 \text{ lb./in.}}{S} \quad 16$$

$$11,250 \text{ lb. wheel load, } E_2 = \frac{80 \times 4.74 \times .22 \text{ lb./in.}}{S} = \frac{83.4 \text{ lb./in.}}{S} \quad 17$$

$$13,390 \text{ lb. wheel load, } E_2 = \frac{80 \times 5.16 \times .24 \text{ lb./in.}}{S} = \frac{99.2 \text{ lb./in.}}{S} \quad 18$$

Subbase modulus can then be obtained by substituting subbase deflection values (Table 10) in equations 16, 17, and 18.

Base Course Modulus

Table 39 summarizes the data required for the modulus calculations.

Using equation 10, and solving for base course modulus, E_1 , then

$$E_1 = \frac{P a}{S} (F_5 \text{ to } \infty - F_{13 \text{ to } \infty}) \quad 19$$

where S is the recorded deflection of the base course (deflection recorded at hole number 1 minus that recorded at hole number 2). Substituting the values from Table 39 in equation 19, the following equations for base course modulus are obtained:

TABLE 38

Dual Wheel Load, lbs.	<u>Subbase Modulus Calculation Data, Depth 13 to 20 Inches</u>				
	Tire Imprint Radius, a, inches	Contact Pressure psi	$F_{13 \text{ to } \infty}$	$F_{20 \text{ to } \infty}$	$F_{13 \text{ to } \infty} - F_{20 \text{ to } \infty}$
6,000	3.45	80	.62	.44	.18
11,250	4.74	80	.82	.60	.22
13,390	5.16	80	.88	.64	.24

TABLE 39

Dual Wheel Load, lbs.	<u>Base Course Modulus Calculation Data, Depth 5 to 13 Inches</u>				
	Tire Imprint Radius, a, inches	Contact Pressure psi	$F_{5 \text{ to } \infty}$	$F_{13 \text{ to } \infty}$	$F_{5 \text{ to } \infty} - F_{13 \text{ to } \infty}$
6,000	3.45	80	.80	.62	.18
11,250	4.74	80	1.06	.82	.24
13,390	5.16	80	1.20	.88	.32

$$6,000 \text{ lb. wheel load, } E_1 = \frac{80 \times 3.45 \times .18 \text{ lb./in.}}{S} = \frac{49.7 \text{ lb./in.}}{S} \quad 20$$

$$11,250 \text{ lb. wheel load, } E_1 = \frac{80 \times 4.74 \times .24 \text{ lb./in.}}{S} = \frac{91.0 \text{ lb./in.}}{S} \quad 21$$

$$13,390 \text{ lb. wheel load, } E_1 = \frac{80 \times 5.16 \times .32 \text{ lb./in.}}{S} = \frac{132.0 \text{ lb./in.}}{S} \quad 22$$

Base course modulus can then be obtained by substituting base course deflection values (Table 10) in equations 20, 21, and 22).

Moduli Values Calculated from Offset Deflections

The procedures used to calculate the elastic moduli of the subgrade, subbase, and base course using offset deflections are identical to those just presented. However, the deflection factors must be found for the proper offset distances. A summary of the data required for the calculations is shown in Table 40.

TABLE 40

<u>Modulus Calculation Data for One and Two Foot Offsets</u>					
Dual Wheel Load, lbs.	Tire Imprint Radius, a, in.	Contact Pressure psi	One Foot Offset F ₂₀ to ∞	One Foot Offset F ₁₃ to ∞ - F ₂₀ to ∞	Two Foot Offset F ₂₀ to ∞
6,000	3.45	80	.36	.06	.248
11,250	4.74	80	.46	.06	.308
13,390	5.16	80	.50	.10	.350

TABLE 4I

Offset Deflection Data Layered System Study

Station	Rear Axle Load, lbs.	Surface		Deflection, .001 inch					
				Hole 1		Hole 2		Hole 3	
		1 ft.	2 ft.	Base		Subbase		Subgrade	
		1 ft.	2 ft.	1 ft.	2 ft.	1 ft.	2 ft.	1 ft.	2 ft.
304	6,000	2.8	0.9	3.7	0.9	4.6	0.9	2.8	0.9
	11,250	7.4	3.7	7.4	3.7	6.5	2.8	7.4	2.8
	13,390	9.3	5.6	7.4	3.7	7.4	5.6	7.4	4.6
275	6,000	3.7	0.9	3.7	1.9	3.7	1.9	2.8	1.9
	11,250	5.6	2.8	7.4	3.7	7.4	4.6	5.6	3.7
	13,390	9.3	4.6	11.1	4.6	11.1	3.5	7.4	4.6
249	6,000	2.8	0.9	4.6	1.9	3.7	1.9	2.8	1.9
	11,250	7.4	3.7	7.4	3.7	7.4	3.7	5.6	3.7
	13,390	8.3	2.8	7.4	2.8	9.3	3.7	8.3	4.6
460	6,000	0	0	2.8	0.9	2.8	0.9	1.9	0.9
	11,250	7.4	2.8	4.6	2.8	7.4	2.8	4.6	2.8
	13,390	7.4	2.8	7.4	2.8	7.4	2.8	3.7	3.7
438	6,000	4.6	1.9	6.5	2.8	4.6	2.8	3.7	2.8
	11,250	5.6	2.8	5.6	3.7	4.6	2.8	3.7	2.8
	13,390	7.4	2.8	5.6	2.8	6.5	3.7	4.6	2.8
426	6,000	6.5	3.7	4.6	3.7	6.5	4.6	4.6	3.7
	11,250	5.6	2.8	5.6	2.8	7.4	3.7	4.6	2.8
	13,390	7.4	4.6	8.3	4.6	7.4	3.7	6.5	4.6

APPENDIX D

Results of Routine Standard Tests on
Subgrade, Subbase, and Surface Samples

FIG.35 GRAIN SIZE ANALYSIS - SUBGRADE SAMPLES, STATION 347

U.S. STANDARD SIEVES

FIG.36 GRAIN SIZE ANALYSIS — SUBGRADE SAMPLES, STATION 249

FIG.37 GRAIN SIZE ANALYSIS - SUBGRADE SAMPLES, STATION 304

FIG.38 GRAIN SIZE ANALYSIS-SUBGRADE SAMPLES, STATION 275

FIG. 39 GRAIN SIZE ANALYSIS-SUBGRADE SAMPLES, STATION 460

U.S. STANDARD SIEVES

FIG. 40 GRAIN SIZE ANALYSIS - SUBGRADE SAMPLES, STATION 453

U.S. STANDARD SIEVES

FIG.4/ GRAIN SIZE ANALYSIS-SUBGRADE SAMPLES, STATION 426

TABLE 42

Atterburg Limits, Subgrade Samples

Station	Depth from Surface, in.	Liquid Limit LL, %	Plastic Limit PL, %	Plasticity Index PI, %
347	23 - 51	25.4	17.5	7.9
	51 - 75	47.3	27.8	19.5
249	23 - 64	31.0	16.1	14.9
	64 - 80	35.6	15.3	20.3
304	23 - 43	31.2	20.7	10.5
	43 - 52	31.7	18.1	13.6
	52 - 70	19.9	15.7	4.2
	70 - 80	34.7	22.8	11.9
275	23 - 48	23.0	17.2	5.8
	48 - 72	40.0	16.9	23.1
	72 - 80	34.9	20.4	14.5
460	23 - 34	26.1	18.2	7.9
	34 - 48	25.9	16.0	9.9
	48 - 66	25.5	18.5	7.0
	66 - 80	28.6	19.7	8.9
453	23 - 31	NP	NP	NP
	31 - 35	NP	NP	NP
426	21 - 25	21.4	15.7	5.7
	25 - 31	22.5	15.6	6.9
438	24 - 32	NP	NP	NP
	32 - 36	NP	NP	NP
	36 - 45	19.2	18.8	0.4
	45 - 66	28.0	18.1	9.9
	66 - 76	20.4	17.2	3.2

TABLE 43

Moisture Content and Dry Density, Depth 80 Inches

<u>Station</u>	<u>Moisture Content Percent</u>	<u>Dry Density lbs/ft³</u>
249	12.8	120.0
304	15.3	114.4
275	9.3	117.5
460	18.6	118.4

FIG.43 SIEVE ANALYSIS SUBBASE
SAMPLES , STATION 347

FIG.44. SIEVE ANALYSIS SUBBASE
SAMPLES, STATION 249

FIG.45 SIEVE ANALYSIS SUBBASE
SAMPLES, STATION 275

FIG.46 SIEVE ANALYSIS SUBBASE
SAMPLES , STATION 460

FIG.47 SIEVE ANALYSIS SUBBASE
SAMPLES, STATION 453

FIG.48 SIEVE ANALYSIS SUBBASE
SAMPLES , STATION 426

FIG.49 SIEVE ANALYSIS SUBBASE
SAMPLES , STATION 438

TABLE 44

<u>Average Bulk Density, Cores, in lbs./ft.³</u>					
<u>Station</u>	<u>Crack Frequency</u>	<u>Total</u>	<u>Core Surface</u>	<u>Component Binder</u>	<u>Base</u>
347	High	144.3	143.5	141.4	148.6
249	Low	141.7	140.1	139.4	144.6
304	High	145.3	141.1	143.5	148.7
275	Low	142.0	141.8	141.3	144.6
460	High	143.6	143.0	141.2	144.6
453	Low	142.2	140.3	136.0	145.4
426	High	140.9	139.6	141.3	143.0
438	Low	141.9	136.0	140.1	144.8

TABLE 45

Average Percent Asphalt, Core Layers

Station	Frequency	Surface	Core Component Binder	Base
347	High	6.2	6.2	4.7
249	Low	6.0	4.9	4.1
304	High	6.2	5.2	4.5
275	Low	6.3	5.1	4.1
460	High	6.4	4.9	4.2
453	Low	6.4	5.0	4.5
426	High	5.9	5.1	5.1
438	Low	6.2	5.9	4.5

APPENDIX E.
Surface Core and Plate Load Tests

SURFACE CORE AND PLATE LOAD TESTS

Triaxial Tests

Triaxial tests were run on most of the 32 cores removed from the test road. Equipment developed by Schaub (27) was used for the triaxial test program. This equipment included a constant temperature room, a loading frame, and triaxial testing equipment permitting a range of lateral pressures.

Total axial strain was measured with a 0.01 mm dial gauge. Strains of the individual layers of the core (surface, binder, and base) were measured with a model M911 Cathetometer, manufactured by the Gaertner Scientific Corporation, Chicago, Illinois. This device, similar to a surveyor's level, measures differences in elevation to the nearest .05 mm.

Preparation of Specimens for Testing

A small portion of the bottom of each core was sawed off to provide a smooth, level surface for testing. Next, the interfaces of the surface and binder and of the binder and base were marked with red paint. The purpose of these marks was to provide a target for the cathetometer for making strain measurements of the individual layers. After marking, the cores were all stored in the constant temperature room until time for testing.

Test Procedures and Results

All tests were of the "quick" undrained type. Loads were applied in increments of 10 psi up to 50 psi and then in increments of 20 psi until the "proportional" or elastic limit was reached. The time interval between load increments was the time required for the axial strain dial to stop all practical movement, or five minutes, whichever came first. Movement of the layer interfaces was measured with the cathetometer.

Figure 50 shows all of the test equipment in operation. A flashlight was used to provide sufficient light to view the mark indicating each layer through the cathetometer.

Stress-Strain Relationships. Table 46 summarizes the stress-strain relationships of each specimen and of its components. A typical set of data in graph form is shown in Figure 51. From this type of curve, a modulus of elasticity value was estimated for each core and its layers. The modulus of elasticity was defined as the deviator stress at the proportional limit divided by the axial strain at that point. The strain used for this computation was corrected for the intercept of the straight line portion of the stress-strain curve with the strain axis.

Statistical Analysis. A two-way classification analysis of variance was performed on the moduli values of the cores and their layers. Significance of a factor was determined by an F test at .05 and .01 levels. For the difference in moduli values as produced by the various layers, the Tukey method for determining a studentized range allowance for a set of means was applied (21). Table 47 presents the moduli

FIG.50 TRIAXIAL TEST EQUIPMENT
IN OPERATION

TABLE 46

Summary of Stress-Strain Data, Triaxial Tests, Cores

Station	Total Core		Surface		Binder		Base				
	P.L.*	%Strain E, 10 ³ psi	P.L.	%Strain E, 10 ³ psi	P.L.	%Strain E, 10 ³ psi	P.L.	%Strain E, 10 ³ psi			
347	69.0	.66	108.3	1.72	6.3	108.3	.70	15.5	127.8	1.06	12.0
	127.8	1.89	185.2	3.11	6.0	147.0	2.88	5.1	147.0	1.89	7.8
249	88.6	.58	108.3	1.46	7.4	147.0	.43	33.2	127.8	.53	24.1
	88.6	.68	147.0	2.00	7.3	127.8	.46	27.8	88.7	.59	15.0
304	69.0	.64	78.8	2.16	3.6	78.8	.39	20.2	78.8	.78	10.1
	69.0	.46	127.8	1.56	8.2	127.8	.54	23.6	69.0	.39	17.7
275	88.6	.52	185.2	2.25	8.2	147.0	.32	45.8	108.3	.78	13.9
	88.6	.78	127.8	1.72	7.4	127.8	.59	21.6	127.8	1.05	12.2
460	127.8	.85	185.2	3.24	5.7	180.0	.42	42.8	108.3	.63	17.2
	108.3	.80	203.9	3.09	6.6	147.0	.69	21.3	108.3	.65	16.7
453	88.6	.70	10.0	.27	3.6	90.0	.59	15.2	50.0	.21	23.8
	49.0	.27	166.2	2.48	6.7	147.0	.76	19.3	108.3	.44	24.6
426	156.6	1.10	231.8	1.91	12.1	113.3	1.31	8.6	194.5	1.10	17.6
	147.0	1.50	108.3	1.64	6.6	88.6	.32	27.7	147.0	2.38	6.2
438	88.6	.43	60.0	.93	6.5	147.0	.44	33.4	127.8	1.11	11.6

* Proportional Limit

FIG.5I STRESS vs STRAIN – CORE 3,STN.249

Table 47. Moduli of Elasticity Values from Core Triaxial Tests
in 10^3 psi

	C-1 Total Core	C-2 Surface	C-3 Binder	C-4 Base	Totals	
	15.3	7.3	33.2	24.1	80.0	
	13.0	7.3	27.8	15.0	63.1	
	17.1	8.2	45.8	13.9	85.0	
Low	11.4	7.4	21.6	12.2	52.6	537.9
Crack	20.6	6.5	33.4	11.6	72.1	
Frequency						
B-1	15.0	6.4	20.0	19.6	61.0	
	12.7	3.6	15.2	23.8	55.3	
	18.2	6.7	19.3	24.6	68.8	
	10.8	3.6	20.2	10.1	44.7	
	15.0	8.2	23.6	17.7	64.5	
	10.4	6.2	15.5	12.0	44.1	
High	6.8	6.0	5.1	7.8	25.7	420.5
Crack	14.1	12.1	8.6	17.6	52.4	
Frequency						
B-2	9.8	6.6	27.7	6.2	50.3	
	15.0	5.7	42.8	17.2	80.7	
	13.5	6.6	21.3	16.7	58.1	
Totals	218.7	108.5	381.1	250.1	958.4	

values arranged for the analysis of variance. Table 48 summarizes the results of the analysis of variance.

Table 48. Analysis of Variance Results of Core Moduli of Elasticity Values

Source of Variation	SS	df	MS	F	F _{.01}	F _{.05}	Signi.
Layer, C	2359.78	2	1179.89	29.95	4.98	3.15	.01 level
Crack Frequency, B	215.36	1	215.36	5.41	7.08	4.00	.05 level
C x B	96.72	4	24.18	0.61	3.65	2.53	NS
Error	2231.02	56	39.84	-	-	-	-
Total	4902.88	63	-	-	-	-	-

The results indicate that core layer affected the moduli values at the .01 level and that crack frequency affected the moduli values at the .05 level. In order to determine which core layers produced moduli of elasticity values significantly different from the other layers, the previously mentioned Tukey method was applied. The average moduli values for the four components were:

$$\text{Entire core, } 218.7/16 = 13.7 \times 10^3 \text{ psi}$$

$$\text{Surface, } 108.5/16 = 6.8 \times 10^3 \text{ psi}$$

$$\text{Binder, } 381.1/16 = 23.8 \times 10^3 \text{ psi}$$

$$\text{Base, } 250.1/16 = 15.6 \times 10^3 \text{ psi}$$

The statistical data required for the Tukey analysis are:

$$\text{EMS (error mean square)} = 39.84$$

$$n = \text{df (degrees of freedom)} = 56$$

$$g (\text{number of data groups}) = 4$$

m (number of items in group) = 16

q (studentized range coefficient for $g = 4$ and $n = 56$
at the .05 level) = 3.74

RF (range factor) = $q\sqrt{EMS/m} = 3.74\sqrt{39.84/16} = 5.9$

Applying the range factor to the ranges of the average elasticity values of the core layer groups:

Entire core, Surface; $13.7 - 6.8 = 6.9$ 5.9 significant

Entire core, Base; $15.6 - 13.7 = 1.9$ 5.9 not significant

Base, Binder; $23.8 - 15.6 = 8.2$ 5.9 significant

On this basis, all groups can be distinguished from each other except entire core values from base values.

Ductility and Penetration

Ductility and penetration tests were run on the asphalt extracted from the core layers. The tests were performed in accordance with "Standard Method of Test for Ductility of Bituminous Materials," ASTM Designation: D113-44 and "Standard Method of Test for Penetration of Bituminous Materials," ASTM Designation: D5-52, with the exceptions that a high vacuum grease was used to lubricate the ductility mold base plates and a constant temperature oven set at 280° F was used to heat the asphalt before pouring it into the ductility molds. The results of these tests are presented in Table 49.

Plate Load Tests

The Indiana State Highway Department conducted plate load tests on the different pavement layers during the construction of the road in 1953. Two years later, in 1955, plate load tests were run on the

Table 49. Average Ductility, cms, and Penetration, .1mm, Core Layers

Station	Crack Frequency	Surface		Core Component Binder		Base	
		Pene.	Duct.	Pene.	Duct.	Pene.	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

pavement surface. Eight different locations were tested covering three of the four basic soil types. These locations did not include any of the locations included in the deflection studies.

The tests on the subgrade and subbase were run in a confined condition, that is a surcharge weight of the subgrade and subbase, and base courses respectively. Plate load tests on the base course and surface were made in an unconfined condition. A twelve inch plate was used for most of the tests run and for all of the tests discussed here.

Modulus of Elasticity

Moduli of elasticity values were computed from test data obtained during the construction period. In this case, since individual layer deflections were not known, two and three layer analyses developed by Burmister (28) were used. The moduli values were all computed at a settlement of 0.05 inches in order to conform as nearly as possible to the deflections found in the deflection study. The results of these calculations are presented in Table 50.

Bearing Pressure Comparison

Tests made on the surface after two years of traffic indicate a general increase in strength. Table 51 compares surface bearing pressures of tests taken in 1953 and 1955. It is believed that most of the bearing pressure increase was due to increase in subgrade strength. This is borne out by the results showing the smallest increase in bearing pressure over an A-1 subgrade.

Table 50. Bearing Pressures and Elastic Moduli Values from 1953 Plate Load Test Data

Station	Soil Type	Avg. Bearing Pressures @ .05 in. psi			Modulus of Elasticity, 10 ³ psi					
		Subgrade	Subbase	Base	Surface	E ₃	E ₂	Subbase	Base	Surface-Base
										E ₁
215	A-6	30	45	60	85	4.24	11.82	12.1	19.1	
264	A-4	19	23	39	65	2.69	4.04	11.4	24.4	
309	A-4	19	31	53	53	2.69	9.42	15.3	12.3	
318	A-4	33	50	67	-	4.68	13.10	13.4	-	
348	A-4	20	33	49	58	2.83	9.60	13.5	14.0	
355	A-4	48	60	64	-	6.80	12.90	9.3	-	
392	A-4	14	28	49	49	1.99	13.95	13.9	11.1	
435	A-1	66	72	85	90	9.34	12.10	15.3	15.3	

Table 5B. Comparison of 1953 and 1955 Plate Load Test Data

Station	Soil Type	Avg. Surface Bearing Pressure, psi @ .05 inch		
		1953	1955	% Increase
215	A-6	85	143	68
264	A-4	65	109	68
392	A-4	49	112	127
435	A-1	90	135	50