FHWA/NJ-82/006
82-007-7702

Status Report

EXPERIMENTAL PAVEMENT PROJECT

Route I-80, Section 5V

Route I-95, Section 1R

September 1981

by

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Division of Research and Demonstration

New Jersey Department of Transportation

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TECHNICAL REPORT STANDARD TITLE PAGE

1. Report No. FHWA/NJ-82/006	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Experimenta Pavement Project		5. Report Date September 1981
		6. Performing Organization Code
7. Author(s) Robert F. Baker		8. Performing Organization Report No. 82-007-7702
9. Performing Organization Name and Address New Jersey Department of Transportation 1035 Parkway Avenue Trenton, New Jersey 08625		10. Work Unit No.
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address Federal Highway Administration 25 Scotch Road Trenton, New Jersey		13. Type of Report and Period Covered Status Report
		14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

See Attached

17. Key Words Pavement Design, Evaluation		18. Distribution Statement	
19. Security Classif. (of this report) None	20. Security Classif. (of this page) None	21. No. of Pages 86	22. Price

Form DOT F 1700.7 (8-69)

ABSTRACT

This report presents annual pavement performance data from nine experimental test sections and analyzes that data using the AASHO Road Test pavement design method and the multilayer elastic Shell pavement design method which was modified by the Hertz-Hogg two-layer theory. The experimental test sections were designed as a satellite study to the 1958-59 AASHO Road Test. The experimental pavement data includes traffic volume and 18 kip loading information, Benkelman beam rebound measurements, rut depths, rolling straightedge data and condition surveys.

The experimental test sections have been subjected to approximately 6.2 million accumulated 18 kip load applications. The 16 year old test sections do not as yet exhibit significant structural pavement failure. However, the typical measures of distress, Benkelman beam rebound measurements, rut depths and pavement cracking are progressing to a point where failure may be imminent.

Benkelman beam temperature correction relationships were determined to normalize rebound data at 60°F. The relationship was not conclusive and will not be applied to the Benkelman beam data.

Water conditions and vehicle spray were observed on the test sections to access the impact of the observed wheelpath rutting. The vehicle spray conditions were not excessive when compared to adjacent pavement conditions.

Recorded rolling straightedge data and associated PSI values do not adequately define the experimental test sections for progressive failure conditions.

Pavement failure is defined and discussed in terms of structural failure of the pavement materials which cannot adequately support heavy truck traffic. Whether or not the pavement fails structurally, a final report will be prepared after the 20 year design life.

The AASHO Road Test regression equations for deflection and structural number do not adequately define observed performance of the experimental test sections. This failure is attributed to applying the regression equations beyond the range of load applications and subgrade conditions from which they were derived.

The multilayered elastic theory of Shell Pavement Design Methods was investigated for potential evaluation of the experimental test sections. The Shell strain and fatigue curves were compared to the experimental test section strain values which were calculated by Bousinesq equations. Shell bituminous stiffness modulus were calculated for the experimental test section bituminous layers. The Shell stiffness modulus was used to calculate effective thickness with the Hertz-Hogg multilayer elastic theory.

FOREWORD

A major aim in preparing this report is to present an overview of the AASHO and the Hertz-Hogg theories for application to the experimental pavement evaluation. It is not the author's intent to thoroughly investigate the full potential of numerous pavement design methods and the multilayer elastic theory. Future reports will investigate additional pavement design methods and will adapt the multilayer equations to the experimental pavement test sections.

Many problems are inherent with the application of pavement design methods to an experimental pavement evaluation. The AASHO method and the Kentucky multilayer elastic method were developed for specific pavements and applications in a specific geographical area. These specific design conditions make it difficult to apply the regression equations to the conditions at the experimental test sections. Revised regression equations and extensive field data are necessary to adapt the AASHO or the Kentucky method to the experimental test section evaluation. The Shell and VESYS methods were developed for a broad range of pavement design applications, materials and locations. However, these two methods are essentially theory from which attempts have been made to correlate with field experience. Similar correlation would be necessary for the experimental pavement evaluation.

Like abstract art which depicts the real world in generalized forms that are suggested by obscure resemblance to natural appearances, the multilayered elastic theory depicts a pavement system in generalized terms that are related to the real world by the perceptions of the user. Although the multilayered elastic theory is based on scientific fact, the utilization of that theory is overshadowed by an imperfect reality of material characteristics.

Material characteristics are the key to good pavement design and the selection of methods to define these characteristics is essential to good material characterization. Asphalt cement (bitumens) properties and qualities vary by producer and by grade. Aggregate qualities and gradations vary significantly with evaluation procedures. The use of material characteristics data requires extreme caution with various pavement design methods. Without adequate material characterizations, pavement design cannot be scientifically derived.

TABLE OF CONTENTS

					Page
I	SUMMARY AND CONCLUSIONS				1
II.	RECOMMENDATIONS				7
III.	EXPERIMENTAL PAVEMENT RESEARCH AND DATA				8
	A.	Introduction			8
	B.	Experimenta Pavement Data			10
		1.	Traffic Data		10
		2.	Benkelman Beam Data		14
			a.	Statistical Test of the Means	18
			b.	Benkelman Beam Rebound Basin	21
			c.	Benkelman Beam Temperature Correction	22
		3.	Rut Depth Data		32
		4.	Condition Survey		35
			a.	Crack Patterns	35
			b.	Vehicle Spray Conditions	40
		5.	Rolling Straightedge Data		40
		6.	Present Serviceability Index		42
	C.	Pavement Evaluation			43
IV.	FLEXIBLE PAVEMENT DESIGN PROCEDURES AND RESEARCH				46
	A.	Introduction			46
	B.	Identification of Pavement Stresses and Properties			47
	C.	Analysis of Results			53
		1.	AASHO Pavement Design Method		53
			a.	Performance Predictions	55
			b.	Thickness Design	58

TABLE OF CONTENTS (CONTINUED)

Page

C. Analysis of Results (continued)

2. She Pavement Design Method 60

a. Fatigue Criteria 61

b. Bituminous Stiffness Modulus 69

c. Thickness Design (Hertz-Hogg Theory) 72

D. Discussion of Non-Destructive Testing Devices 78

REFERENCES 80

APPENDIX A - VESYS STRUCTURAL PAVEMENT DESIGN METHOD 83

APPENDIX B - KENTUCKY-MULTILAYERED ELASTIC ANALYSIS 84

APPENDIX C - DETERMINATION-SUPPORT MODULUS 85

LIST OF TABLES

Page

Table 1 - Traffic Data for Outside Lane January - December 1978 12

Table 2 - Traffic Data for Outside Lane January - December 1979 13

Table 3 - Summation of Twenty-Four Hour Traffic Counts 15

Table 4 - Accumulated 18 kip Axle Repetitions 16

Table 5 - Benkelman Beam Average Rebound 19

Table 6 - Statistical Test of the Significance 20

Table 7 - Mean Test Section - Rebound/Temperature Data 30

Table 8 - Mean Rut Depths 33

Table 9 - Summary of Condition Survey 36

Table 10 - Rolling Straightedge Measurements 41

Table 11 - Present Serviceability Index 44

Table 12 - AASHO Pavement Performance Predictions (Rebound) 56

Table 13 - AASHO Pavement Life Prediction (Structural Numbers) 59

Table 14 - Calculated Strain Values 65

Table 15 - Dynamic Flexibilities 68

Table 16 - Bituminous Mix Stiffness Modulus 71

Table 17 - Effective Thickness 77

LIST OF FIGURES

Page

Figure 1 - New Jersey Experimenta Pavement 11

Figure 2 - Deflection Basin and Rebound Procedure 23

Figure 3 - Benkelman Beam Rebound Basin Profile Eastbound Test Sections 24

Figure 4 - Benkelman Beam Rebound Basin Profile Westbound Test Sections 25

Figure 5 - Mean Pavement Temperature vs. Average Deflection Adjustment Factor 27

Figure 6 - Asphalt-Concrete Pavement Temperatures on January 19, 1965 27

Figure 7 - Average Monthly Temperature 28

Figure 8 - Rebound Adjustment Factor 31

Figure 9 - Rut Depth Measurement Procedure 34

Figure 10 - Crack Patterns 38

Figure 11 - Permissible Asphalt Strain as a Function of Strain Repetition 62

Figure 12 - Permissible Asphalt Strain as a Function of Mix Stiffness Modulus 63

Figure 13 - Equivalent Standard 18 kip Axles 66

Figure 14 - Nomograph for Predicting the Stiffness Modulus of Bituminous Mixes 70

Figure 15 - Stiffness Modulus of Representative Asphalt Mixes as a Function of Temperature 71

I. SUMMARY AND CONCLUSIONS

The nine experimental test sections have been subjected to approximately 6.2 million accumulated 18 kip load applications. The 16 year old test sections do not indicate significant structural pavement failure. However, Benkelman beam rebounds, rut depths and pavement cracking are progressing to a point where failure may be imminent.

Traffic data indicates that 18 kip loads from commercial traffic in the outside lane is approximately 700,000 loads per year, which is a small increase from previous years. At this rate, the test sections will have experienced approximately 9 million loads after a 20 year life

Benkelman beam rebound data reveal a progressive decrease in the structural ability of the test sections to adequately support the accumulated loads without deterioration of the pavement structure. A statistical test of the means indicates that the 1979 rebound values increased significantly from the 1978 values. Benkelman beam rebound measurements were made with the basin measurement method. Rebound basin measurements were graphed to show relatively stiff sections and weak sections. Test Section 6E and 6W (densely graded stone base course) and Test Section 8 and 9W (stabilized base and dry bound macadam) indicate the weakest pavement. Benkelman beam temperature correction relationships were determined to normalize rebound data at 60°F. It appears that the temperature correction data is inconclusive and will not be used to adjust the Benkelman beam rebound values.

Rut depth measurements indicate that Test Section 4E and 4W (gravel stabilized base course) have the greatest rutting with average depths of 1-1/8 inches and 7/8 inches respectively. Most other test sections exhibit 1/2 inch in the wheelpath areas.

It may indeed be that this test section has failed as defined by surface condition failure and to some extent by base course failure which can be characterized by base course remolding. However, cracking, pavement fatigue, and Benkelman beam values indicate that the pavement continues to be serviceable, and continues to support heavy truck traffic. Indeed, the rutting on Test Sections 4E and 4W requires corrective action but not corrective structural action indicated by a deterioration of the base course materials

The crack and condition survey indicates significant outside lane (truck lane) cracking in Test Section 5W (penetration macadam base course). Minor outside lane cracking exists in Test Section 1E and 1W (dry bound macadam), 4W (gravel-slab base), 5E (penetration macadam base course), 6E and 6W (densely graded stone base course) and 9W (4" stabilized base and dry bound macadam). The other test sections do not show cracking in the outside lane

Water conditions and vehicle spray were observed on the test sections. Excessive spray, relative to adjacent pavement conditions, was not noted on any test sections. However, water did accumulate in the wheelpaths of Test Section 4E and in the inside wheelpath (left wheelpath) inside lane of all test sections. The accumulated water in the inside lane must be attributed to poor cross-section drainage and the inside vertical curb.

Ro ing straightedge measurements fai to yield a significant progressive pattern. This failure is attributed to the relatively small test section lengths on which data is collected. Variation in the straightedge data may also be attributed to pavement remolding.

The Present Serviceability Index (PSI) was calculated with the equation recommended by the Federal Highway Administration. The equation is significantly influenced by the rolling straightedge data. The variability of the straightedge data adversely affects the PSI values. Test Section 4E (gravel stabilized base) indicates a PSI value below the established limit of 2.5. At this time, Test Section 4E does not show a significant decrease in serviceability other than problems associated with pronounced rutting. Pavement failure is tentatively defined as a structural deterioration of the pavement materials which can no longer adequately support traffic. Pavement failure is characterized by structural cracking in the base course and disintegration of the surface course. However, rutting must be included in any pavement failure definition, but definitions of severe rutting vary significantly. Regardless of pavement failure conditions, a final report will be prepared after the 20 year design life.

The experimenta test section data were analyzed with the AASHO Road Test Pavement Design Method and the Shell Pavement Design Method (which was modified by the Berger design curves using the Hertz-Hogg theory) to evaluate the ability of each method to define the test section pavement conditions in quantitative terms.

The AASHO pavement life predictions (refer to Table 12, page 56, in text) for the 1979 Benkelman beam rebound values were determined in terms of the accumulated 18 kip loads to failure. The results were compared to the predictions made in the Ninth Interim Report and the actual accumulated 18 kip loading since the Ninth Interim Report. The 1979 estimated remaining 18 kip loadings appear reasonable. The Ninth Interim Report estimates of the remaining 18 kip loadings failed to predict the actual loading for test sections 1 (dry bound macadam), 3 (4" stabilized base), 5 (grave stabilized base), and 6 (densely graded stone). The estimates of 18 kip loadings for the remaining test sections will be evaluated when the test sections actually fail.

The pavement life predictions (refer to Table 13, page 59, in text) for the test section structural numbers were determined in terms of the accumulated 18 kip loads to failure. The estimates for all test sections except Test Section 1 do not appear reasonable. In particular, the estimate for Test Section 6 (densely graded stone base) indicates that the pavement has failed which is not evidenced by actual pavement conditions. The estimate for Test Section 1 appears to be reasonable, when actual yearly 18 kip loads are projected to the twenty year life.

The failure of the AASHO Road Test equations to adequately predict pavement ife is attributed to extrapolation of the equations beyond their limits of data verification. The AASHO Road Test equations were based on 1.2 million 18 kip loadings, relatively thin pavement sections, and relatively higher Benkelman beam deflection-rebound values than the experimental test sections. At this time, the AASHO equations do not adequately define the test section conditions. However, the equations can be modified with test section data to describe the test sections at the time of failure which necessitates failure criteria. The test section data provides a minimum of points but a reasonable estimate of structural numbers and terminal rebound values wi be possible by modifying equation constants. At that time, material coefficients can be established by solving simultaneous equations with the points of failure and by applying appropriate engineering judgment

The Shel Pavement Design Method was reviewed for application to the experimental test sections. Fatigue criteria were evaluated by the Burmister three-layer stress-strain equations which are utilized in the BISAR computer program. However, it is difficult to reference the limited Shel design curves from Cleassen (Ref. 13) to the experimental test sections. In essence, the accurate evaluation of the Shell Method with the experimental test section data cannot be determined without the Shell design manual and BISAR computer program.

Pavement fatigue of the test sections was investigated using the Berger flexibility concept which follows the concept of Shell Pavement Design Method. The Berger design (or Shell) curve uses deflection data to estimate the pavement life in 18 kip loadings. At this time, the use of dynamic flexibilities to predict pavement life cannot be substantiated with the experimental pavement data. Further research and additiona data are necessary for modification of the design flexibility curve with the experimental pavement data.

Bituminous mix stiffness moduli were calculated from the Shell nomograph and equation. The results were not verified by Department laboratory tests. However, the experimental pavement bituminous mix stiffness moduli estimates appear reasonable for use in the design equations and the literature indicates a good correlation between actual laboratory tests and nomograph calculated mix stiffness. The nature of bituminous material properties does not readily provide for the use of bituminous mix stiffness in actual design conditions at this time. The determination and calculation of bituminous mix stiffness are an imperfect science, which requires substantia research.

The appropriate Shell thickness design curves are not available from the literature to accurately evaluate the experimental pavement. However Howkins has substituted the Hertz-Hogg two-layer equations into the Shell Method to evaluate the effective thickness of a bituminous bound layer and additiona equations to evaluate the support material. At this time, the method is unsubstantiated by the experimental pavement data. Inconsistencies exist between the calculated effective thickness and the actual bituminous bound layer thickness for these test sections.

The primary input for the effective thickness equation is the Shell Method bituminous and subbase material characteristics (modulus of elasticity) and the measured pavement response which includes Benkelman beam data and the calculated characteristic length. Both input quantities are inherently sensitive to sma changes. Therefore, it is difficult for equations to represent absolute qualities which in fact are variable It is this reason that makes the effective thickness estimate an almost impossible quantity to accurately define.

Non-destructive deflection testing equipment has been shown to be an effective device for evaluation of pavement condition and pavement overlay design. The Dynaflect, Road Rater and Falling Weight Deflectometer have been found by others to be the most effective devices and might be useful in evaluating the test sections as they get close to their failure conditions. The dynamic deflection equipment could also be a useful tool for evaluation of the Department highway system.

II. RECOMMENDATIONS

The experimental test sections will realistically remain structurally adequate for support of traffic for several more years. However, the evaluation and monitoring of the experimental pavement test sections should be terminated after 20 years of pavement life. The annual monitoring effort should continue to include deflection surveys, rut depths, surface condition surveys, and straightedge measurements. The measurements should be reported annually until the 20 year design life occurs when a comprehensive final report should be completed

In their present form, the AASHO Road Test design model and the Shell Method pavement design mode which is modified by the Hertz-Hogg effective thickness models of pavement behavior do not adequately define the experimental pavement conditions

The AASHO Road Test design equations are based on extensive field research and regression analysis of pavement responses. The equations should be modified to define the experimenta test sections at the time of failure. Initial modifications of equation coefficients can be made for the next annual report. The equation coefficients can be modified and substantiated at the time of pavement failure.

Pavement failure criteria are necessary to define a test section's failure point for application of the AASHO-type regression analysis. Pavement failure criteria should be defined in terms of the structural adequacy of the pavement. Criteria should be developed as part of the next annual report.

The Shell Method is based on the theory of linear elasticity to assess the effective thickness of asphalt bound materials and unbound materials for a three layer system. The method offers flexibility for various material characteristics. The Shell Method should be further researched and investigated for potentia application to the experimental pavement. The Shell BISAR computer program and manual should be obtained for the investigation

The Hertz-Hogg two-layer method which was used with Shell pavement design materials input, utilizes pavement responses to evaluate the effective thickness. This method can be useful with dynamic testing devices for evaluation of pavement structural conditions. Initial estimates of the effective thickness indicate that further research is necessary to modify the expressions for characteristic length and stiffness values for the evaluation of the experimental pavement. These expressions should be modified for the definition of the experimental pavement in the next annual report.

Non-destructive deflection testing devices have been shown to be effective tools for the evaluation of the pavement structural condition and the design of pavement overlays. The use of this equipment should be investigated for pavement evaluation and pavement failure criteria. Such a device could be usefu with the Department's forthcoming pavement management system

III. EXPERIMENTAL PAVEMENT RESEARCH AND DATA

A. Introduction

The New Jersey experimental pavement was constructed between June, 1964 and October, 1964 on an urban section of Route I-80, Ridgefield and Route -95, Fort Lee. Nine experimental test sections were constructed to evaluate the performance of various types of base course materials and to compare these materials with the standard base course

in the AASHO Road Test in 1959 at Ottowa, Illinois. The Ninth Interim Report (Ref. 1) discusses the experimental test section performance

2 million 18 kip equivalent loadings. The report concluded that al test sections were performing satisfactorily although cracking and indications of pavement aging were beginning to appear in all sections.

Purpose: The purpose of this report is to present supplemental data to the Ninth Interim Report and to investigate several pavement design analysis methods for evaluating fina performance data at the future time of pavement failure. The pavement analysis methods wi be assessed in terms of their ability to determine structural layer coefficients for use with the Department's current pavement design procedure (AASHO Interim Design Guide - Road Test).

Experimenta Test Sections: The test sections were constructed in an area of rock cut yielding a stable foundation with essentially uniform bearing capacity. The Department's Bureau of Soils assigned a CBR of 15 soi support value of 6.75, and a regional factor of 2.5 to the area for design of the standard pavement sections.

The experimental test sections were designed to compare New Jersey base course materials relating their performance to the AASHO Road Test structural layer coefficients and to compare the performance of a composite pavement to a conventional New Jersey flexible pavement. One section was designed to compare the standard New Jersey Fine Aggregate Bituminous Concrete (FABC) surface material and the Medium Aggregate Bituminous Concrete

(MABC) surface material. The FABC and MABC surface courses include a top (wearing) course and bottom course (binder unless otherwise noted. The experimenta test sections are shown in Figure 1

Eight experimental test sections were constructed in the eastbound and westbound local lanes of Route I-80 and one experimental test section was constructed in the eastbound loca lane of Route I-95.

Terminology: Bituminous Concrete and Asphalt Concrete (Ref. 2) are high quality, thoroughly controlled hot mixture of asphalt cement and well graded, high quality aggregate, thoroughly compacted into a uniform dense mass. Bituminous and asphalt concrete are used interchangeably in this report

Pavement failure is defined and discussed in Section C - Pavement Evaluation, Page 43.

B. Experimental Pavement Data

1 Traffic Data

The New Jersey Experimental Pavement annual traffic data includes traffic summary sheets, truck classification lane counts and truck weight information from W2 and W4 tables (furnished by the Federal Highway Administration). The data is annually supplied by the Department's Bureau of Data Resources.

Tables 1 and 2 show the average daily traffic (ADT) summary data by month for 1978 and 1979. The 1979 traffic data is not available for June through December. The 1978 data is substituted for the estimated total yearly traffic data used in the calculation of the 18 kip loadings

FIGURE 1

NEW JERSEY EXPERIMENTAL PAVEMENT

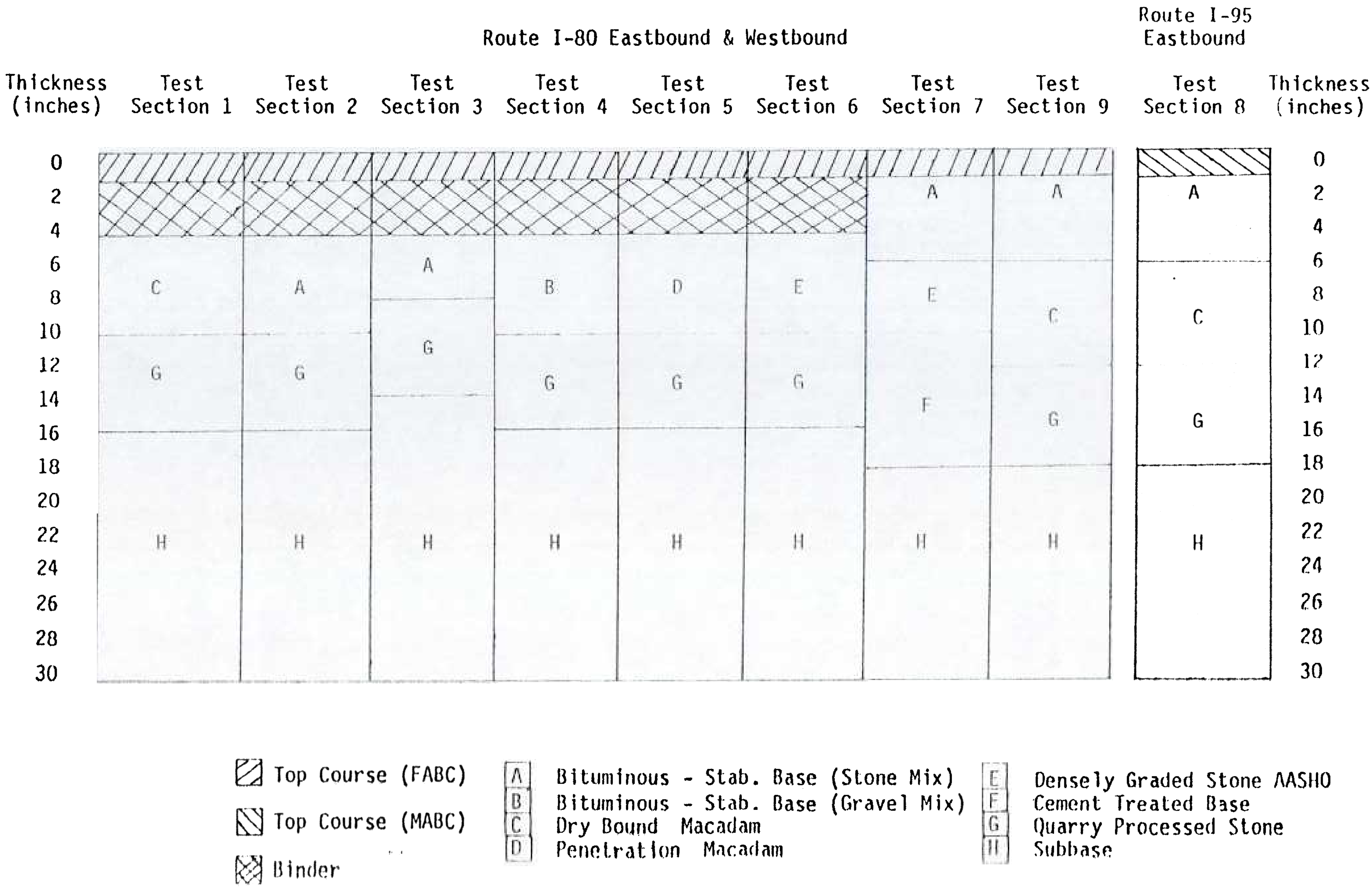


TABLE 1

TRAFFIC DATA FOR OUTSIDE LANE

JANUARY - DECEMBER 1978

Average Daily Traffic	Eastbound Pass. Vehs.	Eastbound Comm. Vehs.	Westbound Pass. Vehs.	Westbound Comm. Vehs.
January	5,318	2,584	5,484	2,339
February	4,695	2,281	5,727	2,443
March	5,511	2,677	6,810	2,905
April	6,155	2,991	6,949	2,964
May	6,260	3,04	7,160	3,054
June	6,493	3,155	7,615	3,248
July	5,986	2,909	6,983	2,978
August	6,227	3,025	6,860	2,926
September	6,277	3,050	6,847	2,921
October	6,303	3,063	7,081	3,020
November	6,086	2,957	6,921	2,952
December	5,992	2,911	6,607	2,818

TABLE 2

TRAFFIC DATA FOR OUTSIDE LANE

JANUARY - DECEMBER 1979

Average	Eastbound		Westbound	
Daily Traffic	Pass. Vehs.	Comm. Vehs.	Pass. Vehs.	Comm. Vehs.
January	5,317	2,607	6,093	2,815
February	5,389	2,642	6,042	2,805
March	5,833	2,859	6,963	3,217
April	5,895	2,890	6,766	3,126
May	5,871	2,878	7,034	3,249
June	6,493*	3,155*	6,806	3,144
July	5,986*	2,909*	6,397	2,955
August	6,227*	3,025*	6,729	3,108
September	6,277*	3,050*	6,764	3,125
October	6,303*	3,063*	6,861	3,169
November	6,086*	2,957*	6,690	3,091
December	5,992*	2,911*	6,426	2,968

*The 1979 monthly average daily traffic data is not available for June through December. The 1978 data is substituted for estimated 1979 18 Kip equivalencies.

Table 3 shows the summation of twenty-four hour traffic counts. The data is an actual count of vehicle by type and classification and is summed here to give the percent trucks

The total yearly traffic data which is used to calculate the 18 kip loading is obtained by multiplying the days per month by the monthly ADT and summed. The twenty-four hour truck percentage is then multiplied by the total yearly vehicles for the tota yearly trucks which is shown in Table 4

Table 4 shows the accumulated 18 kip axle repetitions. The 18 kip axle equivalent factor is obtained directly from W4 tables which are supplied by the FHWA and the Bureau of Data Resources. The factor is multiplied by the total yearly trucks and passenger vehicles to obtain the yearly 18 kip repetitions and it is accumulated in the far right column of Table 4

2. Benkelman Beam

To fac itate road survey safety and ease of measurement, Benkelman beam, rut depth, and straightedge data are limited to the outside (truck) lane in both directions. Pavement design is based on the maximum truck traffic usually in the outside lane.

The Benkelman beam rebound measurements are made at seven stations in each test section. An 18 kip axle load (9,000 lbs. dual wheel load) is used for the rebound measurement procedure. Previous to 1979 the measurement procedure consisted of placing the Benkelman beam probe through the dual tires to a point four feet ahead of the tires. The measurement was made by slowly rolling the truck forward past the beam probe

TABLE 3

SUMMATION OF TWENTY-FOUR HOUR TRAFFIC COUNTS

	Eastbound Outside Lane		Westbound Outside Lane	
	1978	1979	1978	1979
Total Passenger Cars	6,146	6,740	6,952	6,544
Total Trucks	2,992	3,302	2,964	3,026
Total Vehicle	9,138	10,042	9,916	9,570
Percent Trucks	32.7%	32.9%	29.9%	31.6%

TABLE 4

ACCUMULATED 18 KIP AXLE REPETITIONS

EASTBOUND - OUTSIDE (RIGHT HAND) LANE

Date		Traffic		18 Kip Axle Equivalent Factors		18 Kip Axle Equivalent Repetitions				Accumulated 18 Kip Axle Equivalent Repetitions
From	To	Passenger	Commercial	Passenger	Commercial	Passenger	Commercial	Total	To Date[1]	
12/64	5/65	314,321	99,259	.0002	.4279	62	42,473	42,535	5/65	42,535
6/65	11/65	545,669	148,957	.0002	.4279	109	63,739	63,848	11/65	106,383
12/65	11/66	1,112,545	403,486	.0002	.4279('65) .4635('66)	223	214,733	214,956	11/66	321,339
12/66	11/67	1,146,098	422,029	.0002	.4222('67)	228	179,500	179,728	11/67	501,067
12/67	11/68	1,224,729	435,025	.0002	.4266	244	185,427	185,671	11/68	686,738
12/68	11/69	1,316,100	457,138	.0002	.0490	263	187,583	187,846	12/69	874,584
12/69	12/70	1,564,986	457,140	.0002	.3844	313	150,179	150,492	12/70	1,025,076
12/70	12/71	1,260,106	465,844	.0002	.3597	254	167,556	167,810	12/71	1,192,886
12/71	12/72	1,625,894	875,138	.0002	.6397	325	559,823	560,148	12/72	1,753,034
12/72	12/73	1,507,517	959,810	.0002	.6928	302	664,956	665,258	12/73	2,418,292
12/73	12/74	1,547,550	760,981	.0002	.6254	309	475,918	476,227	12/74	2,894,519
12/74	12/75	1,888,106	797,646	.0002	.5662	377	451,627	452,004	12/75	3,346,523
12/75	12/76	1,914,810	1,074,812	.0002	.5662	383	608,558	608,941	12/76	3,955,464
12/76	12/77	2,050,518	1,199,011	.0002	.5878	410	704,779	705,189	12/77	4,660,653
12/77	12/78	2,171,327	1,054,968	.0002	.5878	434	620,110	620,544	12/78	5,281,197
12/78	12/79	2,180,851	1,063,347	.0002	.6524	436	693,727	694,163	12/79	5,975,360

TABLE 4 (CONTINUED)

ACCUMULATED 18 KIP AXLE REPETITIONS

WESTBOUND - OUTSIDE (RIGHT HAND) LANE

Date		Traffic		18 Kip Axle Equivalent Factors		18 Kip Axle Equivalent Repetitions				Accumulated 18 Kip Axle Equivalent Repetitions
From	To	Passenger	Commercial	Passenger	Commercial	Passenger	Commercial	Total	To Date	Total
12/64	5/65	462,069	94,640	.0002	.4279	92	40,496	40,588	5/65	40,588
6/65	11/65	811,171	156,033	.0002	.4279	162	66,767	66,929	11/65	107,517
12/65	11/66	1,672,352	379,330	.0002	.4279('65) .4635('66)	324	174,875	175,199	11/66	282,716
12/66	11/67	1,695,338	416,203	.0002	.4222('67)	329	177,031	177,360	11/67	460,076
12/67	12/68	1,806,367	415,417	.0002	.4266	361	177,061	177,422	11/68	637,498
12/68	12/69	1,923,169	549,739	.0002	.4090	385	225,425	225,810	12/69	863,308
12/69	12/70	2,226,558	511,627	.0002	.3844	490	196,664	197,154	12/70	1,060,462
12/70	12/71	1,994,344	498,148	.0002	.3597	397	179,182	179,579	12/71	1,239,644
12/71	12/72	1,780,119	799,581	.0002	.6397	356	511,487	511,843	12/72	1,751,887
12/72	12/73	2,089,277	870,018	.0002	.6928	418	602,748	603,166	12/73	2,355,053
12/73	12/74	2,207,779	977,993	.0002	.6254	442	611,637	612,079	12/74	2,967,132
12/74	12/75	2,227,054	1,052,919	.0002	.5662	445	596,163	596,608	12/75	3,563,740
12/75	12/76	2,344,012	1,107,259	.0002	.5662	468	626,930	627,313	12/76	4,191,138
12/76	12/77	2,440,751	1,091,465	.0002	.5878	488	641,563	642,051	12/77	4,833,189
12/77	12/78	2,466,851	1,052,194	.0002	.5878	493	618,479	618,972	12/78	5,452,161
12/78	12/79	2,421,549	1,119,031	.0002	.6524	482	730,055	730,537	12/79	6,182,698

A deflection and rebound measurement was recorded. However, only the rebound measurement was utilized for evaluation of test sections. Beginning in the fall of 1979, the Benkelman beam rebound basin procedure was initiated to further define pavement failure parameters (to be discussed later). The procedure consists of marking the pavement at 20 inches (d_{20}) and 40 inches (d_{40}) ahead of the initial probe placement directly under the axle of the dual tires. With the beam stationary the measurements are recorded as the vehicle rolls slowly forward over the marks. The procedure actually measures the rear portion of the deflection basin as shown in Figure 2

Table 5 shows the 1978 Benkelman beam rebound measurements and 1979 rebound basin measurements. It should be noted that the 1978 mean rebound measurements are comparable to the 1979 mean rebound measurements at d_0 (maximum rebound value)

a) Statistical Test of the Means. A statistical test of the means was performed to determine the significance of Benkelman beam increases between 1978 and 1979 and to determine any significance between the 1979 eastbound and westbound measurements. The results are shown in Table 6. The previous rebound data (Ref. 1 and subsequent record reports) shows a gradual increase over several years and periods of leveling off with no apparent statistical increase.

The observed significant difference between the 1978 and 1979 Benkelman beam measurements indicates that the measurements are definitely increasing and probably that the pavement sections have entered a failure or aging mode. A significant increase was noted for all test sections except test section 1W, 3E, 6W, and 9W. The test of significance will be useful for the establishment of failure trends

TABLE 5

BENKELMAN BEAM AVERAGE REBOUND

(Thousandths of an Inch)

OUTSIDE (RIGHT LANE - OUTSIDE WHEEL PATHS

Test Section	October 1978 (Under axle-d_0) Mean	Std. Dev.	October 1979 d_0 Mean	Std. Dev.	d_{20} Mean	d_{40} Mean
1E*	10.0	.63	14.3	1.38	8.6	3.7
2E	8.9	1.07	10.6	.90	6.3	2.9
3E	9.1	.95	.4	3.4	7.4	3.1
4E	10.3	2.69	14.9	2.55	8.6	3.7
5E	9.1	2.54	11.1	1.57	6.0	2.3
6E	12.0	2.31	18.3	3.73	6.8	2.6
7E	8.6	1.51	8.6	0.98	4.0	2.0
8	9.6	0.89	14.8	3.03	5.6	3.2
9E	(Data was not taken due to heavy turnpike ramp traffic)					
1W	14.0	2.58	13.4	3.78	6.8	3.1
2W	6.3	0.75	10.6	0.98	6.6	2.9
3W	9.4	1.51	12.3	1.80	6.9	3.1
4W	9.4	1.90	14.3	5.34	8.6	4.3
5W	12.3	2.14	16.0	2.3	8.3	3.1
6W	17.1	2.27	16.6	2.5	6.6	2.3
7W	6.3	0.75	8.8	1.95	5.1	2.6
9W	11.1	3.02	12.8	1.06	5.7	1.4

*E and W denote eastbound and westbound direction.

TABLE 6

STATISTICAL TEST OF THE SIGNIFICANCE OF DIFFERENCES

BETWEEN 1978 AND 1979 BENKELMAN BEAM MEANS

Test Section	1978 Mean	1979 Mean	t Statistic*	Is Difference Significant?
1E	10.0	14.3	5.327	Yes
2E	8.9	10.6	2.134	Yes
3E	9.1	11.4	1.605	No
4E	10.3	14.9	3.284	Yes
5E	9.1	11.1	1.820	Yes
6E	12.0	18.3	3.902	Yes
7E	8.6	8.6	0	No
8**	9.6	14.8	4.963	Yes
1W	14.0	13.4	0.352	No
2W	6.3	10.6	9.354	Yes
3W	9.4	12.3	3.268	Yes
4W	9.4	14.3	2.532	Yes
5W	12.3	16.0	3.111	Yes
6W	17.1	16.6	0.391	No
7W	6.3	8.8	3.464	Yes
9W **	11.1	12.8	1.559	No

DIFFERENCES BETWEEN 1979 EASTBOUND AND WESTBOUND MEANS

Test Section	Eastbound Mean	Westbound Mean	t Statistic*	Is Difference Significant?
1E & 1W	14.3	13.4	0.652	No
2E & 2W	10.6	10.6	0	No
3E & 3W	11.4	12.3	0.910	No
4E & 4W	14.9	14.3	0.284	No
5E & 5W	11.1	16.0	4.725	Yes
6E & 6W	18.3	16.6	1.019	No
7E & 7W	8.6	8.6	0	No

*Significance indicated at $t_\alpha = 0.05$, $n = 12 = 1.7823$

**Test Sections 8 and 8W are in one direction only.

In Table 6 the means of the 1979 Benkelman beam rebound measurements for eastbound and westbound test sections are not significantly different except Test Section 5E and 5W. This significant difference is attributed to inadequate subbase support in Test Section 5W. It was hypothesized that cracking and relatively high Benkelman beam measurements are caused by poor subbase drainage. The rebound means of the other test sections which are not significantly different indicates that Benkelman beam measurements are consistent between the two roadways and that a reasonable repeatability exists in the measurements on similarly constructed test sections

b) Benkelman Beam Rebound Basin. The Benkelman beam rebound basin is an indicator of pavement structural adequacy. Utah (Ref. 3) reports that rebounds tend to describe structural adequacy with the probable nature of failure being determined by the size and shape of the rebound basin. Utah determined from actua field experience that a pavement is performing adequately when the basin shape is defined by a gently sloping straight line. A pavement is performing inadequately when the rebound basin is defined by a steep sloping line. A pavement defined by a gently sloping line segment and a steep sloping line segment indicates an adequately performing surface course and an inadequately performing base or subbase course. Precise slope values and rebound limits are not available to statistically define pavement conditions. However, the above basin generalizations provide the pavement designer with a means of subjectively evaluating pavement performance. However, any rebound basin analysis must take into consideration the inherent difficulties with basin generalities.

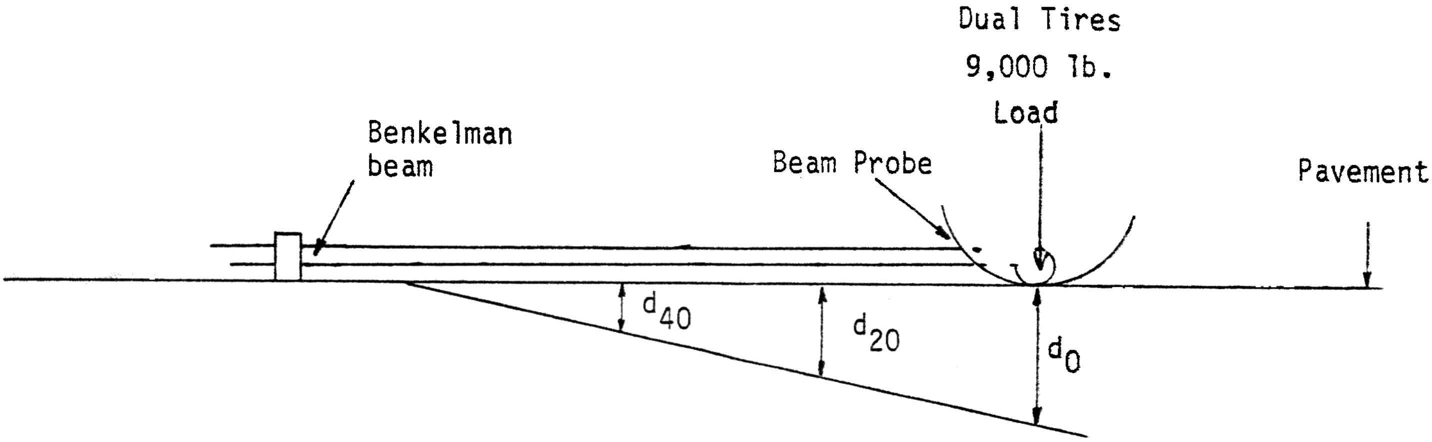
Figures 3 and 4 show the mean test section rebound profiles for the east and westbound roadways. From these profiles it is possible to generalize regarding the pavement structural conditions. The eastbound test section profiles, Figure 3, show steeper slope line segments (d_{20} - d_0) for Test Section 8 (4" stabilized base) and Test Section 6 (densely graded stone base). A "weaker" base is probably indicated for both sections. The westbound test section profiles, Figure 4, do not show any dramatically steeper slopes for line segment (d_{20} - d_0) as observed in Figure 3. However, the profile for Test Section 6 (Figure 2 does exhibit a somewhat steeper ine segment (d_{20} - d_0) than the other test sections. Thus, based on Benkelman beam responses, Test Section 6 apparently has a weaker base course than other test sections.

c. Benkelman Beam Temperature Correction. The AASHO Road Test showed that Benkelman beam deflections (rebound) are affected by seasonal temperature changes but the road test did not develop a regression analysis for the temperature-deflection relationship. The relationship is necessitated by the limited ability to take Benkelman beam data on a year round basis and to relate Benkelman beam data to various design equations. Also, pavement design equations which utilize modulus of elasticity values necessitate accurate pavement temperature and deflection data.

Southgate and Deen (Ref. 4) developed a relationship between mean pavement temperature (average of temperatures at the 0.125, 4.0, and 8.0 inch depths) and deflection (rebound). The relationship adjusts the recorded rebound value to a 60^{O}F reference temperature if

FIGURE 2

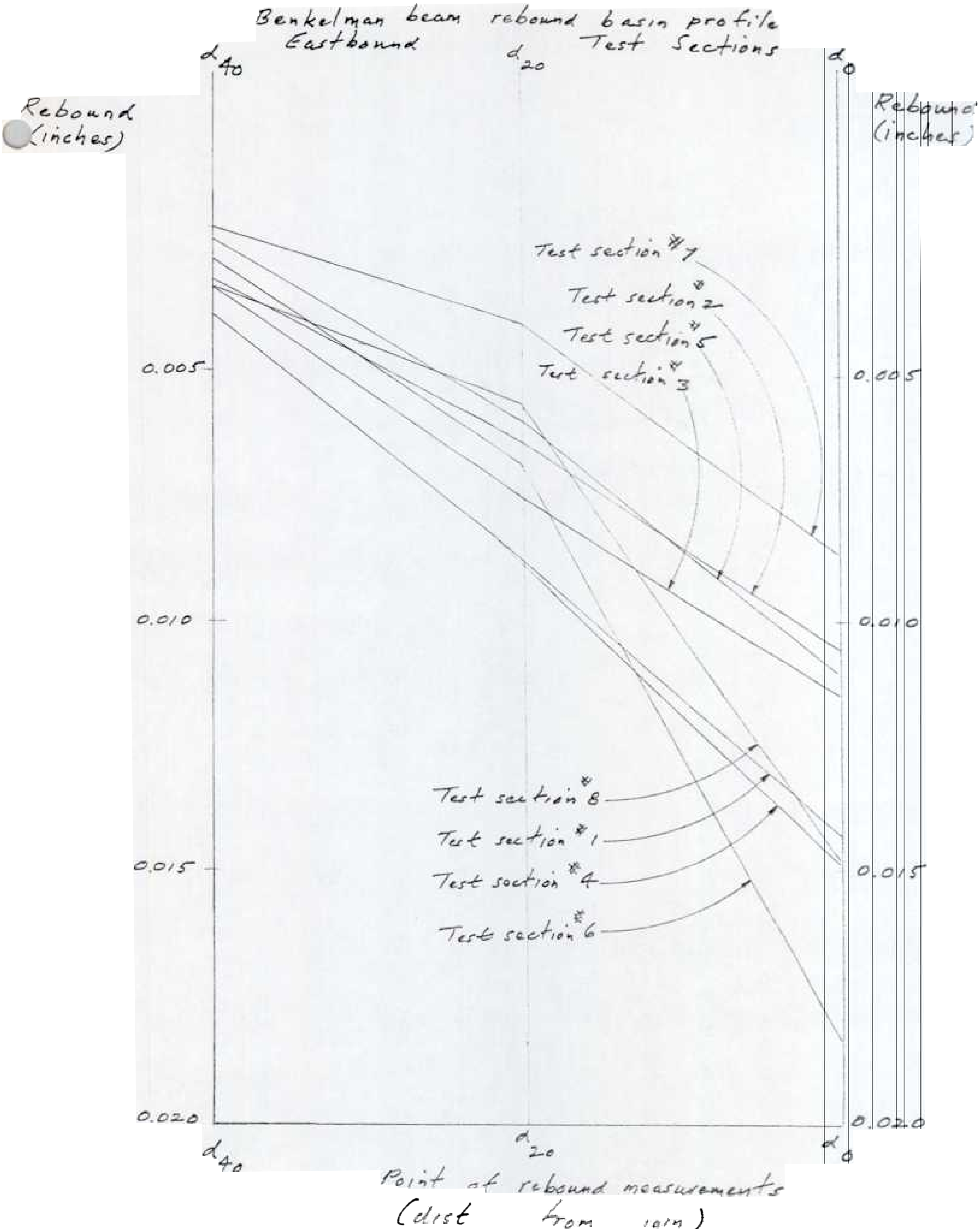
DEFLECTION BASIN AND REBOUND PROCEDURE



d_0 = maximum rebound - initia point (origin) of probe

d_{20} = rebound at 20 inches from d_0

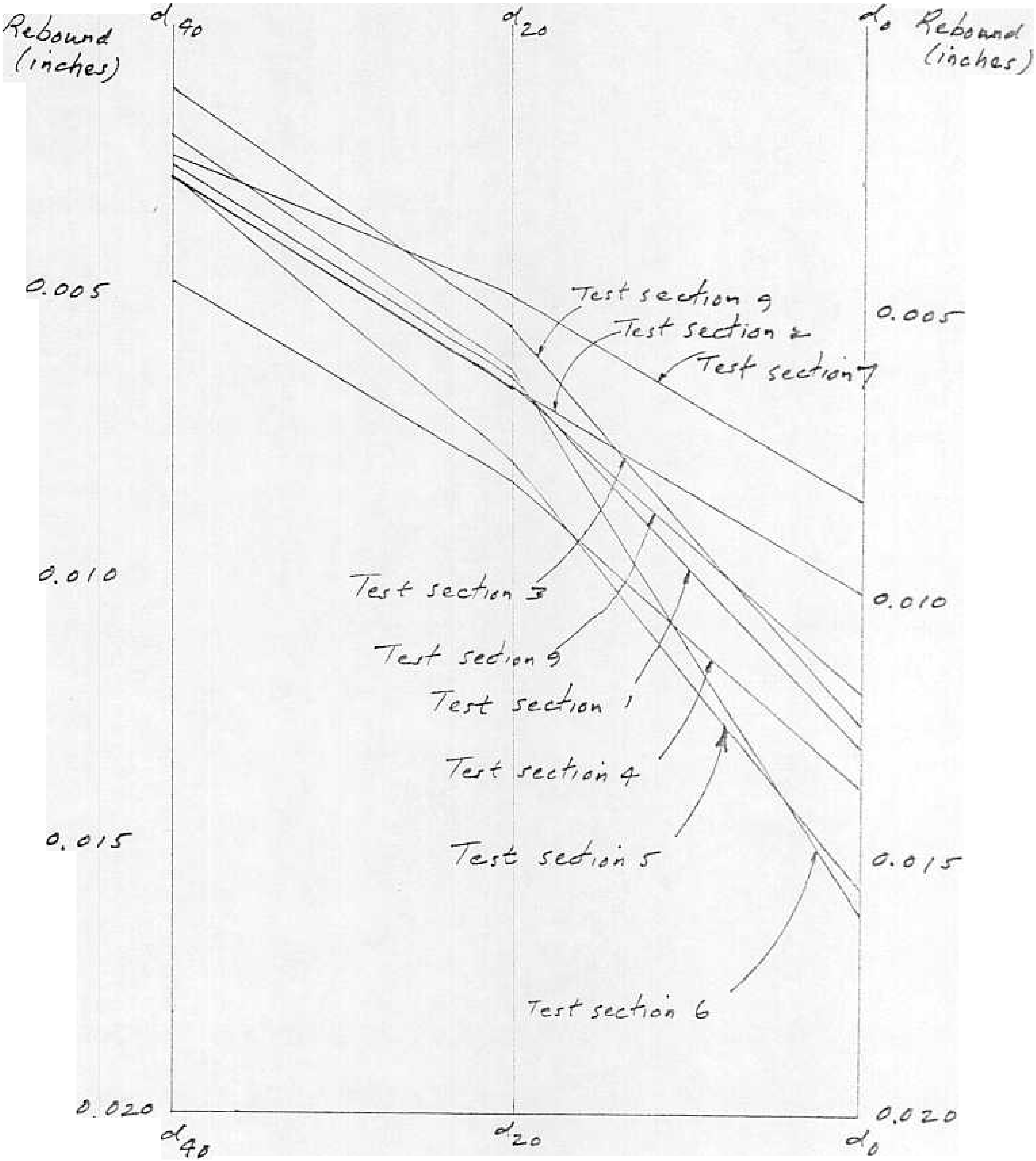
d_{40} = rebound at 40 inches from d_0



Benkelman beam rebound basin profile
Eastbound Test Sections
d_{40}
d_{20}
d_0
Rebound (inches)
Rebound (inches)
0.005
0.010
0.015
0.020
Test section #7
Test section #2
Test section #5
Test section #3
Test section #8
Test section #1
Test section #4
Test section #6
Point of rebound measurements (dist from [illegible])

Figure 4

Benkelman Beam rebound basin Profile
Westbound Test Sections
1979

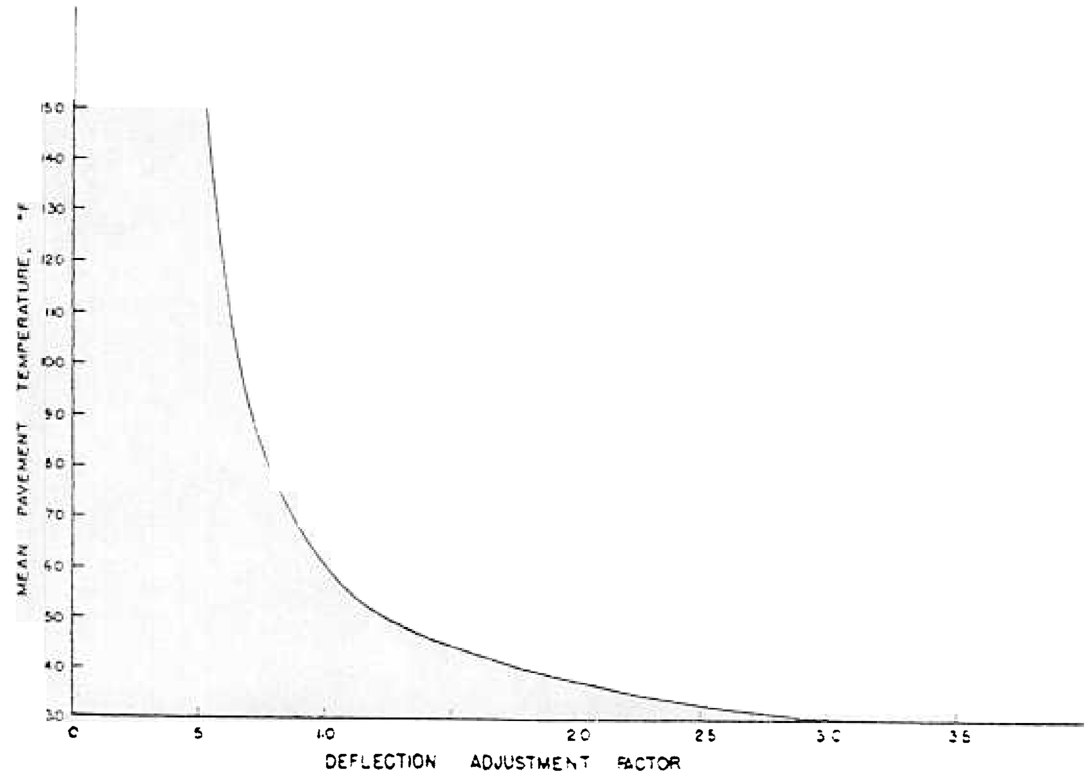


Point of Rebound Measurement
(Distance from Origin)

the mean temperature of the pavement at the time of testing is known or is estimated by the Southgate-Deen method for evaluation of the temperature distribution within flexible pavements. The latter method adjusts mean pavement temperatures for an 8 inch flexible pavement. The deflection adjustment factor, Figure 5, adjusts deflection data for a similarly thin (8 inch) pavement.

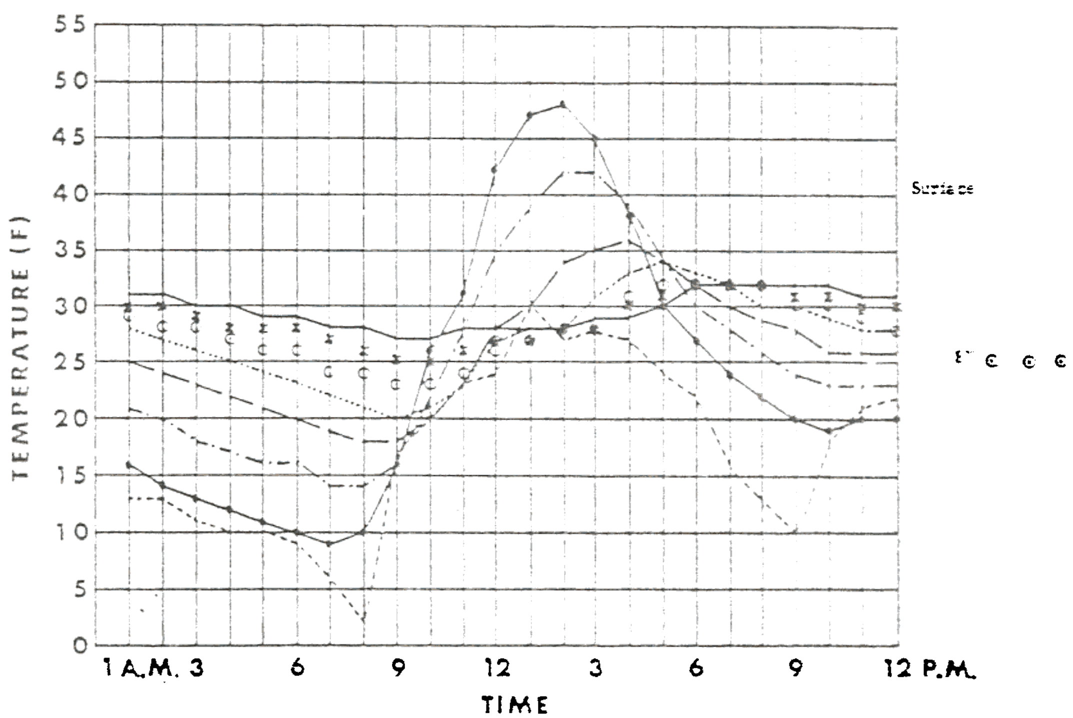
Kallas (Ref. 5) studied the temperature distribution in a 12 inch thick slab of asphalt concrete. He showed that within a 24 hour period the pavement surface temperature varies from 15-20^{o}F less to 15-20^{o}F greater than the temperature at a depth of 12 inches. The temperature becomes more constant as the depth of the pavement increases. Figure 6, from Kallas, illustrates these points. Also, Kallas showed that the average monthly temperature for the pavement surface varies by 4-5% from the average monthly temperature at a depth of 12 inches. The average monthly ambient temperature varies from 2-15^{o}F less than the surface temperature. Figure 7, from Kallas, shows this relationship for June 1964 through May 1965.

Several sources have commented that the Benkelman beam data from the test sections should be adjusted for temperature differentials. The following attempt was made to explore the possibility of Benkelman beam temperature adjustments. Unfortunately, the results are not conclusive and will not be used for Benkelman beam temperature adjustments. The basic problem with the temperature data is an insufficient number of data points for each test section. This problem necessitated pooling temperature-deflection data.



Mean pavement temperature vs average deflection adjustment factor.

FIGURE 5: SOUTHGATE AND DEEN[4] TEMPERATURE DISTRIBUTION WITHIN ASPHALT PAVEMENTS AND ITS RELATIONSHIP TO PAVEMENT DEFLECTION.



Asphalt-concrete pavement temperatures on January 19, 1965.

FIGURE 6: KALLAS[5] ASPHALT PAVEMENT TEMPERATURES

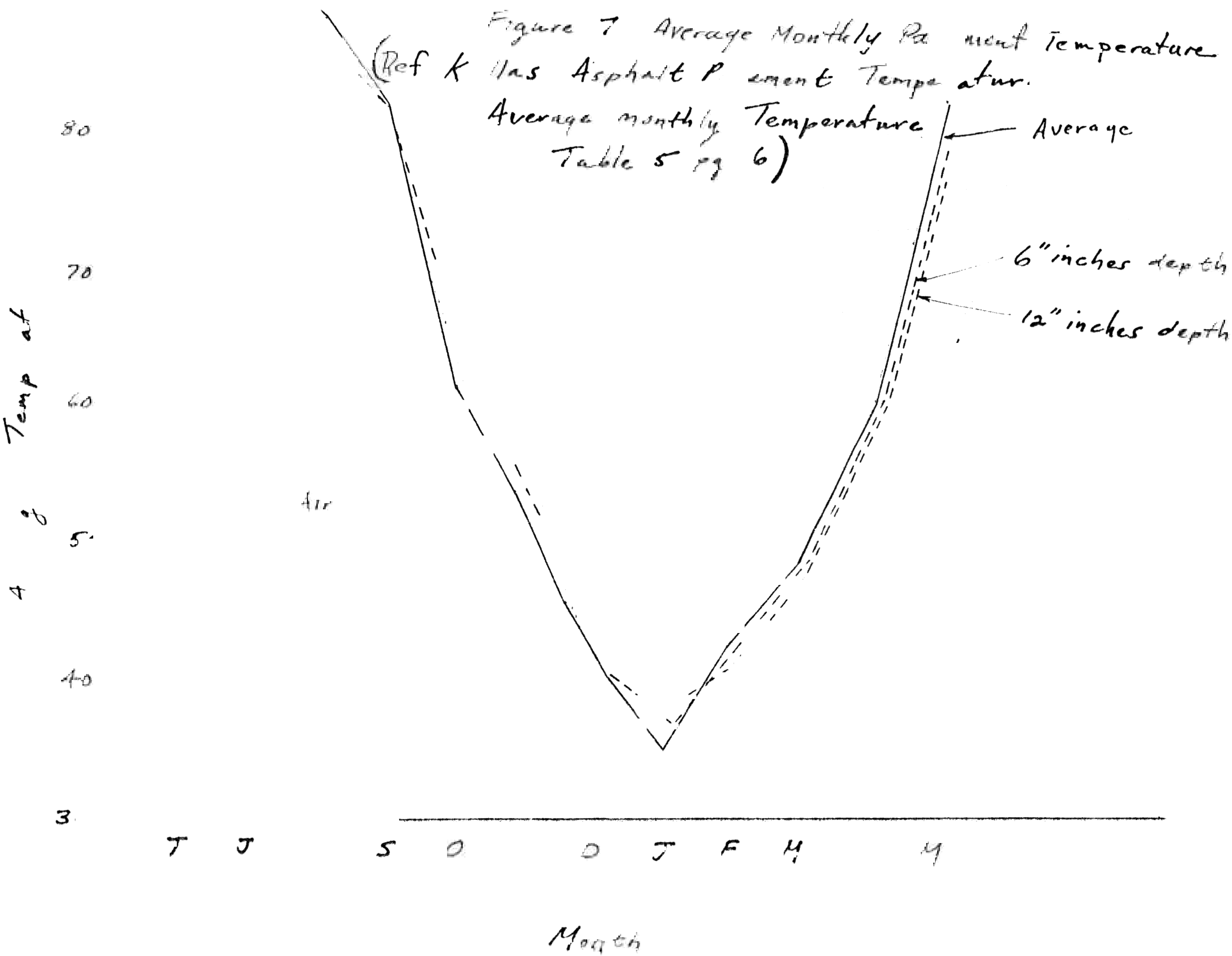


Figure 7 Average Monthly Pa ment Temperature (Ref K llas Asphalt P ement Tempe atur. Average monthly Temperature Table 5 pg 6)

The experimental pavement data available from the Routes I-80 and I-95 project does not include complete pavement temperature data. However, spring 1970 rebound (ambient temperature = 70°F, surface temperature = 82°F) and fall 1970 rebound (ambient temperature = 55°F surface temperature = 58°F) were made to determine the seasonal effect on Benkelman beam measurements. The difference between spring and fall rebound values were calculated and divided by the change in temperature. The mean and standard deviation of this data (Δ rebound)/Δ temperature were obtained by test section and pooled for a final mean and standard deviation. This approach minimized the effect of various pavement sections (thickness and materials) on the rebound-temperature relationship but did not completely remove the bias, because temperature effects a fu bituminous pavement differently than the other base sections. This data is shown in Table 7.

Using the above data, the temperature adjustment factor was calculated using the Southgate-Deen equation:

$$\text{Adjustment factor} = \frac{\text{Rebound at } 60^{\circ}\text{F}}{\text{Rebound at } X^{\circ}\text{F}} \qquad \text{(Equation 1)}$$

Southgate and Deen chose the arbitrary 60°F reference rebound value. The experimental pavement rebound adjustment factor curve is shown in Figure 8. The lack of temperature data points limits the curve accuracy to only those two points from actual experience. Extrapolation between the points and beyond the points is purely theoretical and is based on the Southgate-Deen equation which was applied to a relatively thin flexible pavement. As can be seen in Figures 6 and 7 by Kallas, the average monthly temperature is not a fair indicator of the actual

TABLE 7

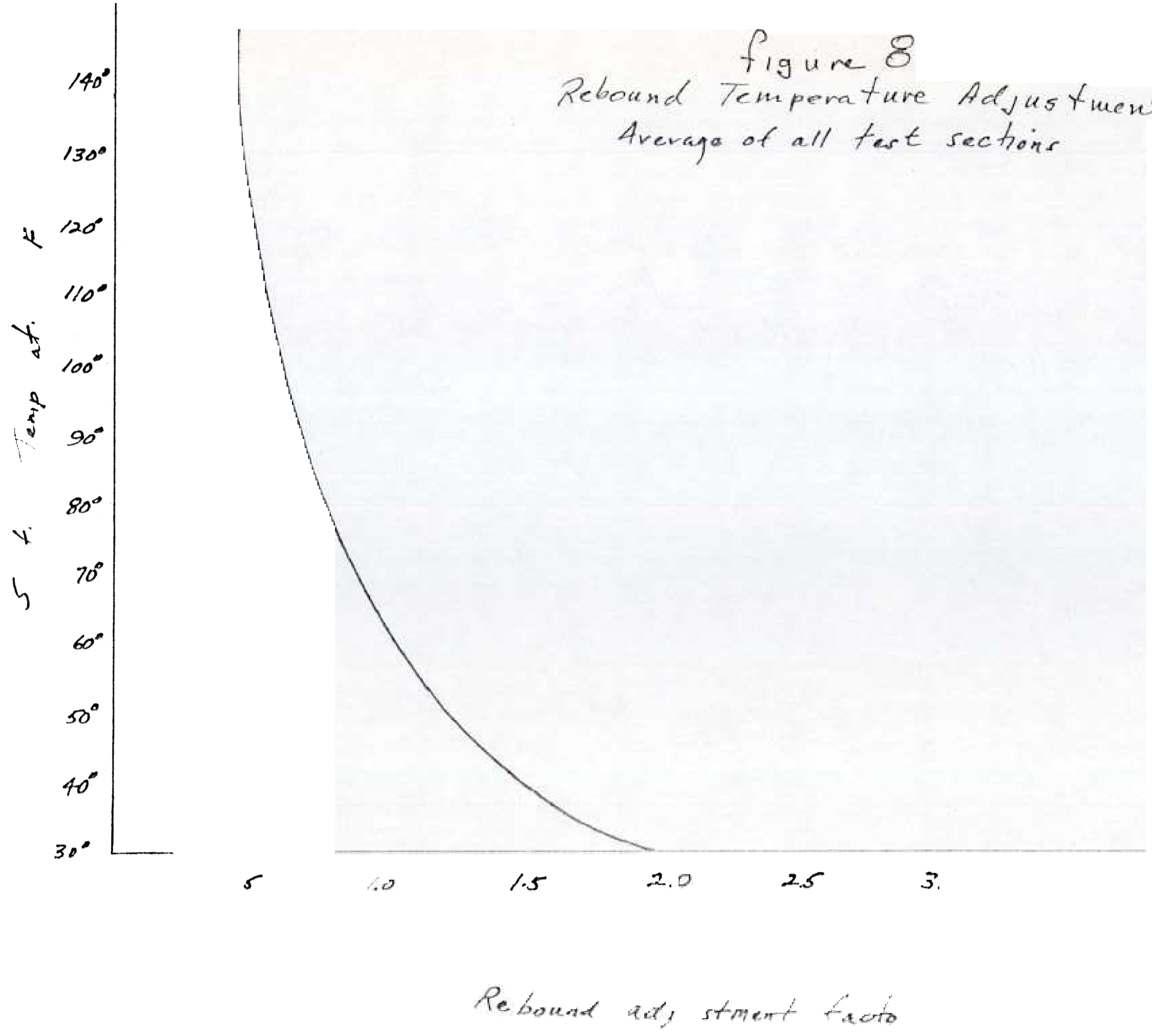
MEAN TEST SECTION REBOUND/TEMPERATURE DATA

Test Section	Ambient Δt = 15° Mean	Ambient Δt = 15° Stand. Dev.	Surface Δt = 24° Mean	Surface Δt = 24° Stand. Dev.
1E	-0.204	0.130	-0.131	0.081
2E	-0.152	0.210	-0.095	0.131
3E	-0.162	0.246	-0.113	0.143
4E	-0.333	0.139	-0.208	0.087
5E	-0.076	0.130	-0.047	0.081
6E	-0.400	0.196	-0.250	0.123
7E	-0.019	0.120	-0.012	0.075
8	-0.027	0.287	-0.083	0.156
9E	-0.201	0.226	-0.131	0.141
1W	-0.314	0.147	-0.196	0.092
2W	-0.295	0.305	-0.184	0.190
3W	-0.257	0.309	-0.065	0.250
4W	-0.314	0.083	-0.196	0.052
5W	-0.390	0.342	-0.244	0.214
6W	-0.400	0.252	-0.250	0.158
7W	-0.152	0.403	-0.095	0.252
9W	-0.247	0.451	-0.166	0.280
Pooled Values	-0.249	0.253	-0.145	0.167

figure 8

Rebound Temperature Adjustment

Average of all test sections



24 hour pavement temperature variability. In essence, the adjustment factor is an over-simplification of a complicated pavement temperature structure.

An additional area for discussion of temperature-Benkelman beam-temperature relationships is the effect of spring rains on the Benkelman beam data. It is possible that moisture in the base and subbase decreases the pavement's strength and increases Benkelman beam measurements, which gives bias to the spring data.

3. Rut Depth Data

The mean rut depth data of both wheelpaths for the eastbound and westbound outside lanes are given in Table 8.

Rut depth measurements are made by placing a ten foot straightedge across the wheelpath as shown in Figure 9. The rut depth is the distance from the lower edge of the straightedge to the pavement surface. The most severe rutting is associated with Test Sections 4E and 4W (gravel stabilized base). The least rutting is associated with Test Sections 1E (dry bound mac.) and 9W (4" stabilized base and dry bound mac.) In previous years, the least rutting was associated with the Test Sections 7E and 7W (cement treated). These sections continue to exhibit a low degree of rutting in comparison to the other sections. Pavement remolding and minor errors in the method of measurement probably contributed to the variation in rut measurements.

TABLE 8

MEAN RUT DEPTH (INCHES)

OUTSIDE (RIGHT) LANE - BOTH WHEEL PATHS

Test Section	October 1978 (Average)	October 1979 (Average)
1E*	0.48	0.43
2E	0.58	0.53
3E	0.55	0.53
4E	0.85	1.12
5E	0.49	0.52
6E	0.54	0.53
7E	0.49	0.48
8	0.47	0.48
9E	(Data not taken due to heavy turnpike ramp traffic)	
1W	0.51	0.56
2W	0.69	0.67
3W	0.75	0.69
4W	0.86	0.86
5W	0.62	0.56
6W	0.63	0.53
7W	0.64	0.64
9W	0.50	0.45

*E and W denote eastbound and westbound direction.

Rut Depth

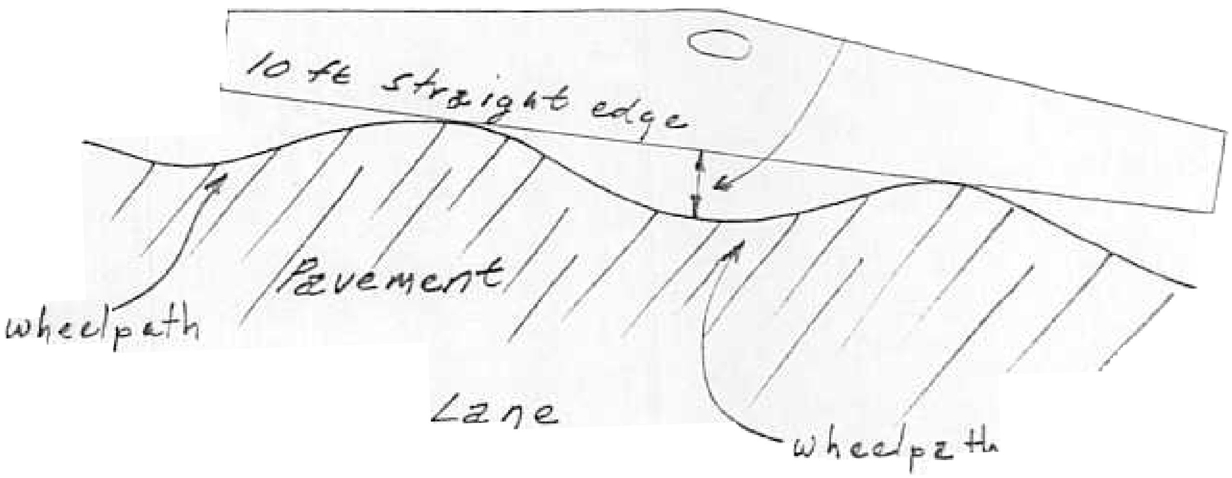


Figure 9

Rut Depth measurem t Procedure

4. Condition Survey

A detailed condition survey of surface cracking was conducted on all test sections. Table 9 shows a subjective analysis of pavement cracking by test section and by lane. No significant change in cracking (since 1978) was noticed in all test sections except Test Section 7E (cement treated base). Fine (1/8 inch or less) transverse cracks began to appear two years ago. Initially five transverse cracks about 12" - 18" long appeared at the outside edge of the outside lane. Now there are twelve cracks about 24 inches long and 1/8 inch wide. At this time the twelve transverse cracks which are distributed over the 500 ft. test section do not significantly affect the pavement's performance. However, the cracks may indicate future pavement failure conditions.

a. Crack Patterns. Crack patterns are shown in Figure 10. They are typical of cracking in the test sections. Test sections are identified by crack pattern type in Table 9, Summary of Condition Survey. At this time no conclusions will be drawn regarding a correlation between crack patterns and pavement failure conditions.

Cracking in all test sections except 5W has not reached a serious stage and does not appear to affect the Benkelman beam measurements (deflections) because cracking does not predominate in the outside lane Inside and center lane cracking can be attributed to excess surface water which permeates into the pavement. In essence, poor surface drainage and the lack of pavement underdrains appears to be the cause of the observed pavement cracking

TABLE 9

SUMMARY OF CONDITION SURVEY

ROUTE I-80, SECTION 5V AND ROUTE I-95, SECTION 1R

JANUARY 1980

Test Section	Percentage of Cracking Inside Lane	Center Lane	Outside Lane	Description of Pavement Condition
1E	65%	60%	20%	Small (1/8" wide) longitudinal cracks in both wheel-paths. (Type 1)*
2E	5%	None	None	Very small less than 1/8" wide) cracks in both wheelpaths. (Cracks too small to classify.)
3E	30%	10%	None	Small longitudinal cracks in both wheelpaths. (Type 1)
4E	50%	5%	None	Small diagonal cracks in the right wheelpath of the inside lane. Small diagonal cracks in the left wheelpath of the center lane. (Type 5)
5E	50%	40%	5%	Small longitudina' cracks in both wheelpaths. (Type 1)
6E	5%	5%	5%	Very small cracks in both wheelpaths. (Cracks too small to classify.)
7E	None	None	None	Small transverse cracks appear in the outside lane. (Type 4)
9E	20%	5%	None	Small cracks in both wheelpaths. (Type 2)

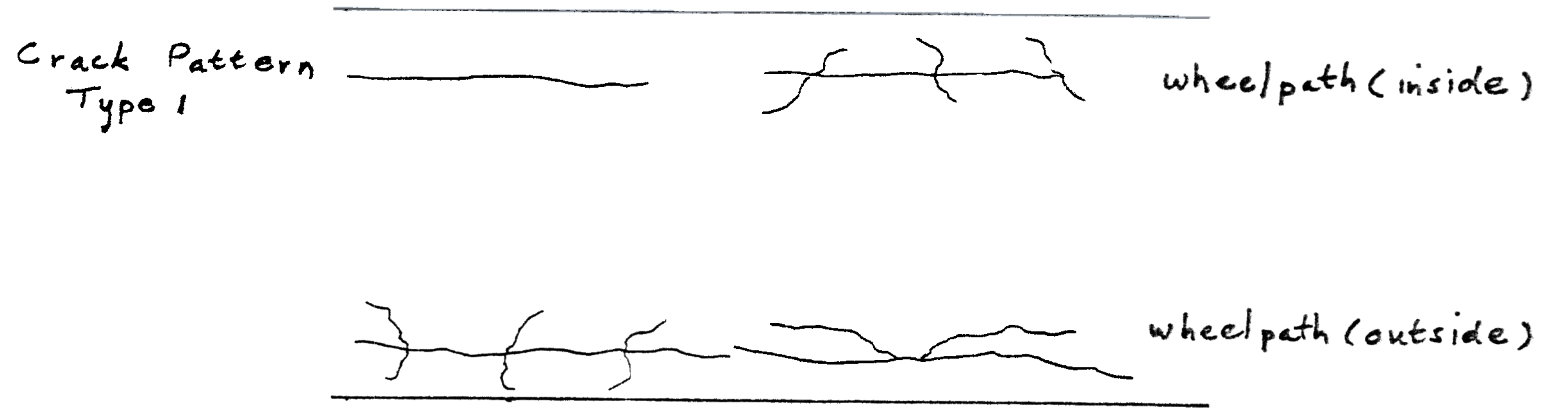
*Crack pattern type is shown in parentheses and described in Figure 10.
Refers to lanes where cracking is indicated.

TABLE 9

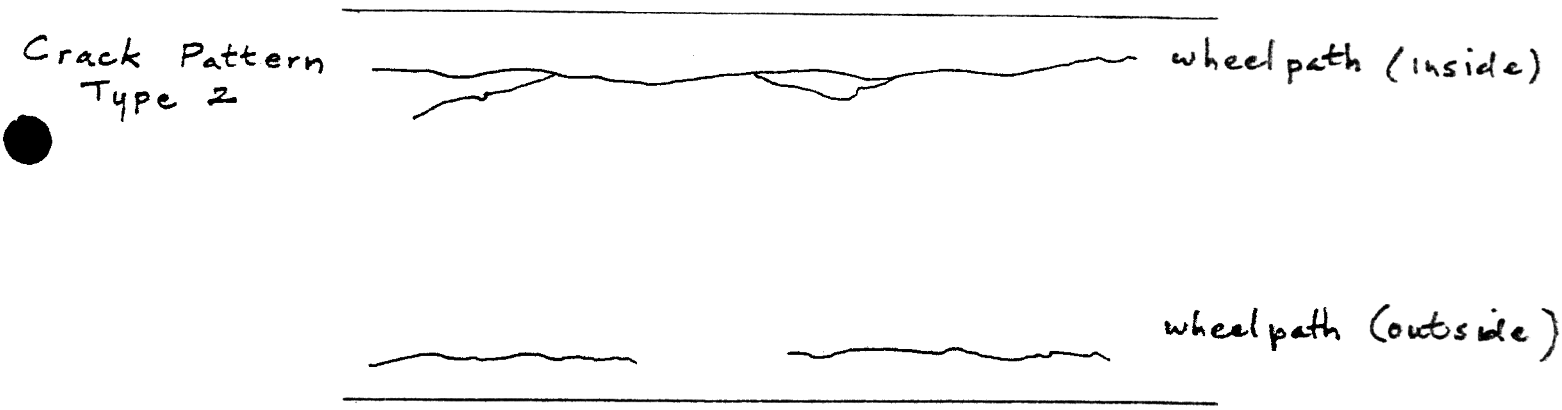
JANUARY 1980

Test Section	Percentage of Cracking Inside Lane	 Center Lane	 Outside Lane	Description of Pavement Condition
1W	90%	15%	5%	Small (1/8" wide) transverse and longitudinal cracks. (Type 2 and Type 4)
2W	40%	5%	None	Small transverse and longitudinal cracks. (Type 2 and Type 4)
3W	25%	60%	None	Small longitudinal cracks in both wheelpaths. (Type 1)
4W	5%	5%	5%	Small longitudinal cracks in both wheelpaths. (Type 2)
5W	95%	95%	40%	Small (1/8" wide) and medium (1/4" wide) longitudinal cracks in both wheelpaths. Cracking has not progressed since last survey. (Type 3)
6W	60%	95%	5%	Small longitudinal cracks in both wheelpaths. (Type 1)
7W	20%	20%	None	Small transverse cracks in the outside lane, inside lane and center lane. (Type 2 and Type 4)
9W		95%	20%	Small longitudinal cracks in both wheelpaths. (Type 1)

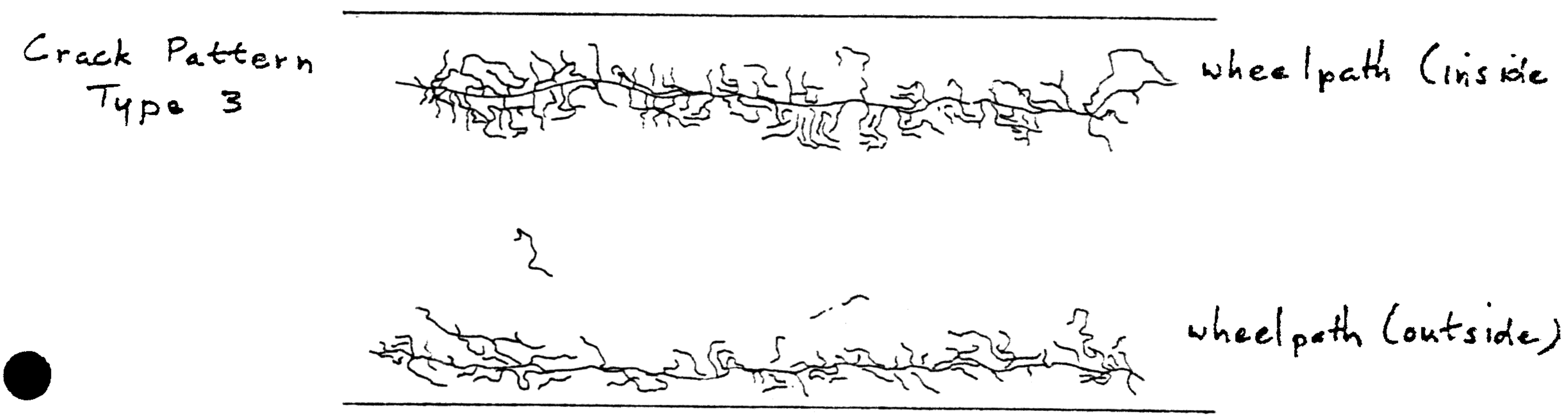
figure 10
Crack Patterns



Lane (cracks about 1/8" wide)
wheelpath long. and diagonally accross wheelpath.



Lane (cracks about 1/8" wide)
Long. wheelpath Cracks



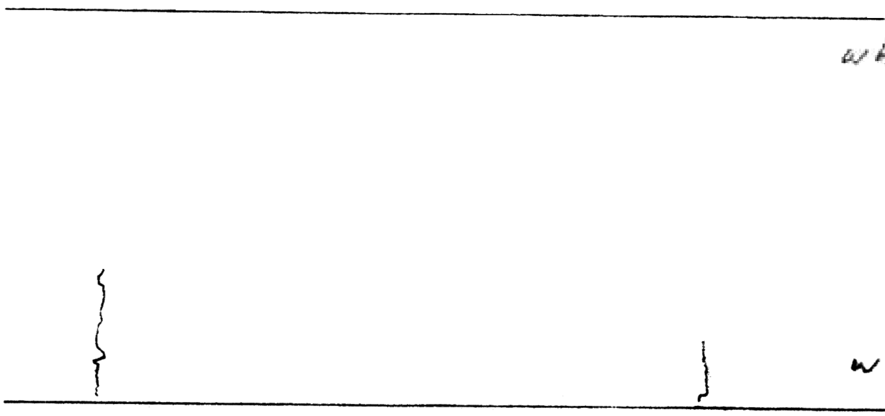
Lane (cracks 1/8" to 1/4" wide)
Random long. and allegator cracking

C k P tte ns

igure

Crack Patt
Type 4

wh path (id



whee path (outs

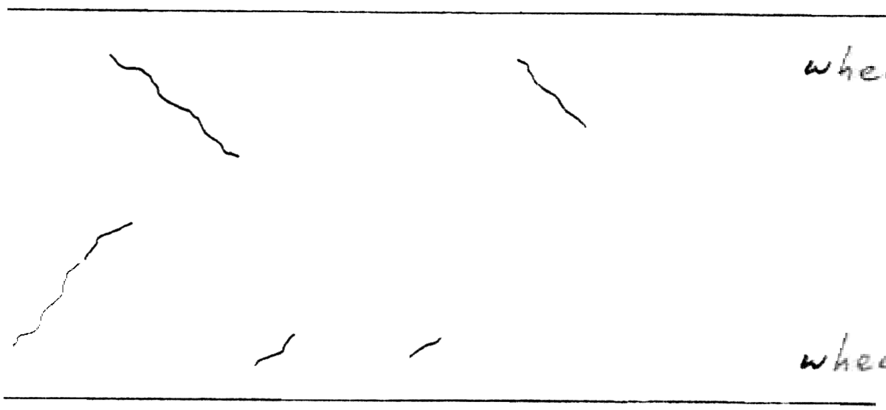
L

Tr s Crack g (less than 1/8 wide)

Crack Patter
T pe 5

wheelpath (inside)

Traffic



wheelpath (outside)

L

Diagonal C acking (1/8 wid)

b. Vehicle Spray Conditions. Vehicle spray conditions on the test sections were observed during a heavy rain storm in April 1980. A full size Plymouth sedan was driven over the test section to experience vehicle spray and vehicle handling characteristics. The survey determined that water was channeled in both wheelpaths of the outside (right) lane of Test Section 4E and all inside (left) wheelpaths of all inside lanes in all test sections. Vehicle spray conditions on the inside lanes and outside lanes where water accumulated moderately affected the driver's ability to see and steer the vehicle. However, no hydroplaning sensation was observed at 50 mph

The vehicle spray conditions on road sections before and after the test sections were not significantly different from the test section. Spray conditions on the test sections could not be rated unacceptable. It follows then that the level of rutting in Test Section 4 is not sufficiently pronounced to be considered an impairment to vehicle safety

The channelized water in Test Section 4E is attributed to the severe rutting and remolding of the gravel stabilized base. Water in the inside lane, inside wheelpath of all test sections is attributed to inadequate drainage and cross-slope. Inside lane rutting and an 8" vertical curb probably contribute to this channelized water.

5. Rolling Straightedge Data

The rolling straightedge or roughness data from the outside lane, both wheelpaths, is shown in Table 10 for April 1979 and May 1980. Although many pavement design methods place great pavement roughness as

TABLE 10

ROLLING STRAIGHTEDGE MEASUREMENTS

Outside Lane

Inner and Outer Wheelpath

Test Section	April 1979 Percent Defective Length	May 1980 Percent Defective Length
1E*	2.0	3.2
2E	2.5	4.4
3E	1.6	3.2
4E	8.3	8.6
5E	1.9	2.8
6E	4.9	4.0
7E	2.2	2.6
9E		
1W	1.0	1.6
2W	3.8	2.0
3W	5.9	4.1
4W	1.6	0.8
5W	3.7	2.6
6W	3.6	1.9
7W	3.9	2.5
9W	9.1	5.8
8	9.4	7.3

*E - Eastbound, W - Westbound

an indicator of pavement serviceability. The sixteen years of data and experience with the test sections does not yield a significant pattern of roughness changes to adequately describe pavement serviceability or failure. Therefore, any evaluation of the test sections' rolling straightedge data should be approached with extreme caution.

The failure of the straightedge data to yield a significant failure pattern is attributed to the short length of the test sections from which data is extrapolated into percent defective. Variations in the straightedge data may also be due to pavement remolding as shown by rutting measurements which fluctuate from year to year. However at this time, correlation has not been established between straightedge data fluctuation and rut depth fluctuations.

6. Present Serviceability Index

The Present Serviceability Index (PSI) was developed at the AASHO Road Test to define the terminal point of the pavement users concept of pavement serviceability. The PSI equation and AASHO Road Test are discussed later in the report. The New Jersey experimental pavement PSI numbers were calculated by the following equation recommended by the Federal Highway Administration:

$$PSI = 11.29 - 4.11 \text{ Log } R - 0.01\sqrt{(c+p)} - 1.23\ RD^2 \qquad \text{(Equation 2}$$

Where:

R = Roughness units (inches per/mile

RD = Mean rut depth in inches

c+p = Cracking and patching factor

The cracking and patching factor is disregarded in the calculation for the PSI value because patching is nonexistent and cracking has not reached a significant level in the outside lane of most test sections. Only Test Section 5W has notable cracking in the outside lane. However, the cracking adds only a very small contribution to the PSI value.

In Table 11, the Present Serviceability Index values are shown for the outside lane of the test sections. The AASHO Road Test established the terminal point of 2.5 for major highways. The terminal point was derived from correlations and subjective analysis of the AASHO pavement sections.

For evaluation of the New Jersey experimental pavement test sections, the terminal PSI value of 2.5 wi   not be used to establish a terminal point. The present New Jersey PSI equation, which uses only roughness and rut depth parameters, does not appear to adequately define a failure condition for the New Jersey experimental pavement. The PSI equation should be adjusted at the time of pavement failure for the conditions at the test sections, and expanded to include the additional parameter of pavement deflections.

C. Pavement Evaluation

Pavement evaluation is discussed with the intent of developing a pavement failure definition from three sources of information (a) AASHO (Ref. 11), (b) Shell (Ref. 13 and 14) and (c) Canada (Ref. 26). The main objectives of pavement evaluations are to determine whether the intended function and expected performances are being achieved, and

TABLE 11

PRESENT SERVICEABILITY INDEX

Outside Lane

Test Section	1978	1979
1E	3.14	3.07
2E	2.97	2.85
3E	3.10	2.95
4E	1.99	1.31
5E	3.12	3.00
6E	3.27	2.89
7E	3.10	3.08
8	2.53	2.66
1W	3.19	3.08
2W	2.65	2.87
3W	2.38	2.63
4W	2.55	2.63
5W	2.78	2.97
6W	2.79	3.08
7W	2.73	2.88
9W	2.53	2.82

to provide information for planning future rehabilitation for pavements. The major types of pavement evaluation for the above objectives can involve one or more of the following:

1. Structural deterioration which occurs when the pavement will not support heavy loads

2. Roughness which is related to riding comfort, safety, and vehicle operation

3. Skid resistance which defines surface texture and vehicle safety

4. Surface conditions such as rutting which relate to hazardous water conditions and vehicle handling.

A discussion of pavement evaluation should include quantitative values for pavement materials characterizations and relationships Pavement materials have been characterized by modulus of elasticity values which can be obtained from laboratory and field data. However the New Jersey experimental pavement data will probably be unable to develop accurate modulus of elasticity values for materials. The AASHO regression approach to structural numbers will be necessitated by the New Jersey experimental pavement data. The specific method by which these numbers are obtained is important because the method determines the value of the structural coefficients.

In this report a structural failure method will be evaluated, rather than a skid resistance or Present Serviceability approach to pavement failure. Therefore, pavement failure is defined as a structural deterioration of the pavement materials which cannot adequately support heavy truck traffic. Pavement failure is characterized by structural cracking in the base course and high deflection values. Rutting, which in many methods is associated with surface conditions, is also considered structural or base course failure.

The following sections of this report will discuss pavement evaluation in terms of structural failure as defined above. The regression analyses of the AASHO Road Test and the multilayered elastic theory of the Shell method wi be investigated for their potentia for evaluation of base course materials and structura coefficients. Also, the two and three layer equations which are used in the Shell method are discussed for background information of the Shell multilayer elastic theory A discussion of the Berger modifications of the Shell pavement life graphs and thickness equations are presented in terms of pavement structura deterioration which emphasizes pavement deflections and stress-strain relations.

The evaluation of the experimental pavement test sections will probably terminate with a final report after 20 years of service. It appears that sufficient data and information wi be available to project failure by cracking, rutting and deflection at 20 years for a test sections regardless of the actual degree of failure

IV. FLEXIBLE PAVEMENT DESIGN PROCEDURES AND RESEARCH

A. Introduction

The New Jersey experimental test sections were conceived fundamentally in accordance with the AASHO Road Test design procedure developed prior to the experimenta test sections. Since that time new pavement design and analysis procedures have been developed utilizing the multilayer elastic theory. Also, material characteristics research has developed testing procedures to better define flexible pavement materials. Therefore, it is prudent to consider the revelancy of the new procedures to evaluate the experimenta test sections.

The purpose of this part of the report is to investigate some of the newer design/analysis procedures for evaluation of the experimental test sections and using the experimental test section data, compare the results of these procedures with the results of the AASHO Road Test. A iterature search of prospective design procedures produced an excessive quantity and variety of information. Therefore, it was expedient to eliminate repetitive procedures and discuss only those procedures which could possibly be applied to the New Jersey experimental test section data

B. Identification of Pavement Stresses and Properties

It must be realized that the following multilayered elastic theory deals with idea materials and ideal conditions, which are imperfectly satisfied in nature. Judgment as to the validity of the theory and application of the theory should be based on actual performance and experience. However, the theory can be used to explain the real phenomena for which it is in reasonable agreement

The classical assumptions (Ref. 6) of the theoretical multilayered elastic system are (1 the material properties of each layer are homogeneous; (2) each layer has a finite thickness except for the lower layer and all are infinite in the latera direction; (3) each layer is isothropic;

(4) fu friction is developed between layers at each interface;
(5) surface shearing forces are not present at the surface; and
(6) the stress solutions are characterized by the material properties represented by Poisson's ratio and an elastic modulus.

The layered pavement design theory (Ref. 7) was derived by Love in 1923 and Timeshenko in 1934 in the form of stress and displacement equations of elasticity for the three dimensional condition. The theory, which is used for most linear elastic pavement design methods explains norma stresses acting perpendicular to the element face and shearing stresses acting parallel to the face. The theory distinguishes three properties of material behavior response. These responses are the relationship between stress and strain (linear or nonlinear), the time dependency of strain under a constant stress leve (viscous or non-viscous) and the degree to which the materia can rebound or recover strain after stress removal (plastic or elastic)

The analytical expression for stress, strain, and deflection by Boussinesq (Ref. 6), Burmeister (Ref. 7), and Hogg and Hertz (Ref. 8, 9 and 10) wil be analyzed in Subsection C of this section for their ability to define the experimental pavement conditions.

The Boussinesq one layer expression defines the vertical stress at any depth below the soil surface due to a point load at the surface as follows:

$$\sigma_Z = K\frac{P}{Z^2} \qquad \text{(Equation 3)}$$

where:

K = factor for radial distance from point load
Z = depth
P = total load

In the above expression, the vertical stress is dependent on the depth and radial distance and is independent of the properties of the transmitting medium. The Boussinesq expressions are applicable for subgrade stress, strain and deflection studies where the pavement does not contribute any significant deflection component to the total surface deflection. The Boussinesq equation was further refined by Ahlvin and Ulery (Ref. 6) and tables were developed for stress-strain solutions to one layer equations.

The vertica and horizonta strain equations (after Ahlvin and Ulery) are:

Vertical strain: $\epsilon_Z = 1.5 \frac{P}{E_1} B$ (Equation 3a

Where: P = unit load (psi)

E_1 = modulus of elasticity of the pavement layer

B = one layer elastic function value

Horizontal strain: $\epsilon_r = 1.5 \frac{P}{E_1} C$ (Equation 3b)

Where: P = unit load (psi)

E_1 = modulus of elasticity of the pavement layer

C = one layer elastic function value

The above strain equations assume that Poissons' ratio equals 0.50, and that the values of "B" and "C" are functions calculated and tabulated by Ahlvin and Ulery (Ref. 6).

The Burmeister two layer expressions represent flexible pavements composed of layers that decrease in modulus of elasticity with the pavement's depth. Deflections are obtained for ideal homogeneous conditions as follows:

$$\Delta = 1.5 \left(\frac{pa}{E_2}\right) F_2 \qquad \text{(Equation 4}$$

Where: p = unit load on circular plate

a = radius of plate

E_2 = modulus of elasticity of lower layer

F_2 = dimensionless factor dependent on the ratio of the moduli of elasticty of the subgrade and pavement, and depth of radius ratio.

The Burmeister two layer stress-influence curves (Ref #6, page 43) are analyzed for the "F_2" factor.

The three layer analytical expressions for stresses and displacements were developed by Burmeister, Acum, Fox, Jones and Peattie (Ref. 6). The five representative stresses in the three layer system are solved by stress factor values developed by Peattie and Jones. The five stresses are:

$\sigma_Z 1$: vertical stress at interface 1

$\sigma_Z 2$: vertical stress at interface 2

$\sigma_r 1$: horizontal stress at the bottom of layer 1

$\sigma_r 2$: horizonta stress at the bottom of layer 2

$\sigma_r 3$: horizonta stress at the top of layer 3

Assuming Poisson's ratio, μ is 0.5 for all layers, several types of strain may be computed from the calculated stresses and the equations for strain. The horizontal strain at the bottom of layer 1 may be computed as follows:

$$\epsilon_{r1} = \frac{1}{2E} \quad \sigma_{r1} - \sigma_{Z1} \qquad \text{(Equation 5)}$$

The vertica stress solutions were obtained by Peattie (Ref. 6, page 48) in graphica form and the horizontal stress solutions were obtained by Jones (Ref. 6, page 64) Interpolation of stress factors is permitted but extrapolation is not allowed.

The mechanics of determining various stresses, strains and deflections within a multiple structure are intended to provide a quantitive evaluation of the fundamentals for pavement design. However, it is difficult to confirm the validity of stress factors and the multilayer expressions for use with the experimental pavement. It is necessary to establish expressions which can be utilized to define existing pavement conditions and which can be simply verified.

The Hertz theory and Hogg model of an elastic plate on an elastic foundation was adapted for pavement design by Wiseman (Ref. 8, 9, 10) The Hogg two layer model expresses the asphalt bound layer thickness in relation to the dynamic asphalt concrete modulus of elasticity, Poisson's ratio and flexural rigidity which defines the structura support of the underlying material. The basic expression for thickness is:

$$t = \sqrt[3]{\frac{D\ 12(1-\mu^2)}{E_1}} \qquad \text{(Equation 6)}$$

Where:

μ = Poisson's ratio

E_1 = the dynamic modulus of elasticity for the asphalt bound pavement layer

D = the flexural rigidity of the pavement which is dependent of the characteristic length and subgrade (supporting structure) modulus of elasticity and is expressed:

$$D = \frac{E_s \ell^3}{(1+\mu)} \qquad \text{(Equation 7)}$$

Where:

E_s = the subgrade modulus

μ = .5 for most soils

ℓ = the characteristic length

$$\ell = m\ (1-{}^{dr}/do)^{-n} - c \qquad \text{(Equation 8)}$$

Where:

dr = the deflection (rebound) at distance "r" from maximum deflection do (point zero)

m, n & c = coefficients from curve analysis

The following curve fitting coefficients are suggested by Howkins:

r	m	n	c	dr/do max. validity
50 mm	56	0.46	50	0.88
100 mm	78	0.60	60	0.70

The characteristic length (ℓ) depends on the size of the deflection bowl which is defined by the measured relationship of the maximum deflection (do) and the deflection (d_{50}, d_{100}) at 50mm or 100mm from the maximum deflection. The dynamic modulus of elasticity (E_1) of the asphalt concrete is determined from laboratory tests but, in this study, the Shell Method will be used to estimate suitable values of the asphalt modulus of elasticity which varies with the temperature, time and asphalt and aggregate content.

The Hertz-Hogg model as interpreted by Wiseman is a two layer system that relates thickness to material characteristics. The system assumes stress-strain relationships similar to the Boussinesq and Burmeister expressions. However, imitations are inherent in the use of this simplified model to describe the behavior of a complex structure such as a flexible pavement.

C. Analysis of Results

The VESYS Structura Pavement Design Method, Kentucky Multi-layered Elastic Design Method, AASHO Pavement Design Method and the Shell Pavement Design Method were reviewed for prospective evaluation procedures. The VESYS and Kentucky method are discussed in the Appendix. The AASHO and Shell Methods are discussed in the following paragraphs.

1. AASHO Pavement Design Method

The AASHO pavement design procedure (Ref. 11 & 12) is the result of road tests conducted in Ottawa, inois in 1958-1959. The design method is based on the road user concept of pavement failure. This concept determines the required strength of a pavement structure necessary to withstand a given number of load applications before the pavement performance reaches a given terminal Present Serviceability Index (PSI) The required pavement strength is given numerically in terms of a Structural Number (SN), which is the accumulated value of each pavement layer thickness being multiplied by its layer strength coefficient.

Thus, SN is given by:

$$SN = a_1 d_1 + a_2 d_2 + \ldots\ldots a_n d_n \qquad \text{(Equation 9)}$$

Where:

a_n = layer strength coefficient (nondimensional)

d_n = layer thickness (inches)

n = number of layers above the subgrade

The layer strength coefficients of materials were determined empirically from regression equations at the Road Test. Various material strength coefficients were determined for different pavement thickness and the fina values were averaged for each material.

The Present Serviceability Index is made up of a correlation of user concept of pavement serviceability and various physical measurements of the pavement. The AASHO correlation equation for flexible pavement is:

$$PSI = 5.03 - 1.91 \log (1 + SV) - 1.38 \, RD^2 - 0.01 \, (c + p)^{1/2} \qquad \text{(Equation 10)}$$

Where:

SV = slope variance, a measure of longitudinal roughness

RD = average rut depth

c + p = area of class 2 and 3 cracking plus patching per 1000 ft^2

The AASHO flexible pavement design equations were developed from an analysis of the effect of structural design (component thickness and material type) and loading upon the performance of the road test sections.

The fundamentals of the AASHO design procedure are expressed in the basic equation which was developed at the AASHO Road Test. The equation is:

$$\log W_{t18} = 9.36 \log (SN + 1) - 0.20 + \frac{\log (4.2 - P_t)/4.2 - 1.5)}{0.40 + \ 1094/(SN + 1)5.19} +$$

$$\log \frac{1}{R} + 0.372 \ (SS - 3.0) \qquad \text{(Equation 11)}$$

Where:

W_{t18} = the number of 18 kip single axle applications to time t

P_t = the terminal PSI

SN = the structural number of the pavement

R = the regional factor which represents a correction factor for various climates

SS = soil support and is correlated with different soil strength tests, such as CBR.

Equation 11 predicts pavement life for the climatic, subgrade, material and traffic conditions experienced at the AASHO Road Test Although the equation accounts for various climate and soil conditions, the regiona factor and soil support numbers are broad generalizations of conditions which do not necessarily exist in various states. The regional factor and soil support value have no rational basis and are arbitrary values intended to be established by individual authorities

a. Performance Predictions. The estimated remaining 18 kip loadings for the test sections were calculated using Equation 10 and the 1979 Benkelman beam rebound values (average rebound + 2 standard deviations). The results are shown in the second column of Table 12. At this time it is impossible to verify these predictions. However, the predictions appear reasonable and will be verified as the test sections progressively deteriorate.

TABLE 12

PAVEMENT PERFORMANCE PREDICTIONS FROM

THE AASHO PAVEMENT PERFORMANCE EQUATION FOR REBOUND MEASUREMENTS

$$\text{Log } W_{2.5} = 7.98 + 1.72 \log L - 3.07 \text{ Log } d_{fn}$$

Combined Eastbound and Westbound Test Sections	1979 Estimated Remaining 18 Kip Loadings (Reb. + 2σ)	9th Interim Report Estimated Remaining Accumulated 18 Kip Loadings (Reb. + 2σ)	Actual Accumulated 18 Kip Loadings Since 9th Int. Rep.*
1	1,750,000	4,200,000	5,000,000
2	4,800,000	8,700,000	5,000,000
3	2,350,000	4,000,000	5,000,000
4	1,050,000	5,800,000	5,000,000
5	2,650,000	3,500,000	5,000,000
6	500,000	2,100,000	5,000,000
7	7,750,000	16,200,000	5,000,000
8	1,300,000	7,300,000	5,000,000
9	2,200,000	6,500,000	5,000,000

*Test sections have experienced a total of 6.2 million 18 kip loads.

In the Experimental Pavement inth Interim Report, pavement performance predictions were also estimated with the AASHO Road Test regression equation for Benkelman beam rebound values and accumulated 18 kip loadings to failure. The predictions from Equation 10 are shown in the third column of Table 12 with the accumulated 18 kip loadings (column four) since the predictions were made in that report. The accumulated 18 kip loadings since the Ninth Interim Report exceed the pavement life predictions for test sections 1, 3, 5, and 6. At this time, the pavement sections do not indicate imminent structura failure. It seems unlikely that the AASHO predictions can accurately define the performance life of other experimental pavement test sections (Test Section 2, 4, 7, 8, and 9).

The apparent failure of the AASHO equation to predict pavement life is attributed to the inability to accurately extrapolate data beyond the accumulated 1.2 million 18 kip loadings of the Road Test and the inherent variability in Benkelman beam measurements. Benkelman beam accuracy was tested by Bush[18] and compared to other non-destructive testing devices. It was determined that, when compared to a deflection standard, the Benkelman beam rated the highest percent error.

The AASHO design equations were derived by averaging regression relationships from relatively thin pavement sections after 1.2 million 18 kip loads. At this time it appears that the AASHO relationship for Benkelman beam deflections and accumulated 18 kip loads is not reliable for the thicker pavement sections and the higher accumulated loads experienced on the experimental test sections. Also, the experimental test sections are constructed on a rock cut which yields a stiffer subgrade than the AASHO Road Test. In essence, additional regression equations are necessary to evaluate the test sections. However, there is a imited amount of data for the test sections which restricts the application and accuracy of any regression analysis of the test section data.

b. Thickness Design: The AASHO Road Test equation (Equation 11) to predict pavement ife in terms of the number of allowable applications of an 18 kip load is given for the structural number (SN), regional factor (R), and soil support (SS). The structural numbers (SN) are derived from "The AASHO Interim Guide for the Design of Flexible Pavement Structures".

The 6.75 soil support value and 2.5 regional factor, which were selected by the pavement designer for the experimental test sections, were substituted in Equation 11 for pavement life predictions. The result was distorted by a multiple of 10. For example, with one test section, the estimate of pavement life without the soi support value and regional factor gives a pavement ife of 8.7 million 18 kip loads while an estimate of 86 million 18 kip loads is obtained when these parameters are included. The distortion occurs because the predicted values are extrapolated well beyond the limits of the data collected of the Road Test

Using the general form of Equation 11, i.e., without the soil support and regional factor terms, the pavement ife predictions were made for each test section; the results are shown in Table 13. The pavement life predictions, except Test Section #1, appear to be entirely unreasonable. The pavement ife prediction for Test Section #1 (dry bound macadam base course could be reasonable because the predicted

TABLE 13

AASHO PAVEMENT LIFE PREDICTIONS

PAVEMENT LIFE IN ACCUMULATED 18 KIP LOADS FOR STRUCTURAL NUMBERS

Test Section	AASHO Structural Number	Pavement Life Accumulated 18 Kip Loads	Actual Loadings To Date
1	5.34	8,700,000	6,200,000
2	6.78	53,800,000	6,200,000
3	6.12	24,300,000	6,200,000
4	6.18	26,100,000	6,200,000
5	5.94	19,100,000	6,200,000
6	4.98	5,300,000	6,200,000
7	6.92	63,400,000	6,200,000
8	6.00	20,800,000	6,200,000
9	6.00	20,800,000	6,200,000

life of 8.7 x 10^6 loads is somewhat above the present accumulated 6.2 x 10^6 loads. The prediction for Test Section #6 (densely-graded stone base course) indicated that the pavement has failed which is not the case. The prediction for the other test sections appears to be entirely too high to be practical. In essence, the general AASHO Road Test equation and the modified equation which considers soil support and regional differences cannot accurately predict pavement life beyond the Road Test experience. Extrapolation of the AASHO equations must be confirmed by additional data.

2. Shell Pavement Design Method

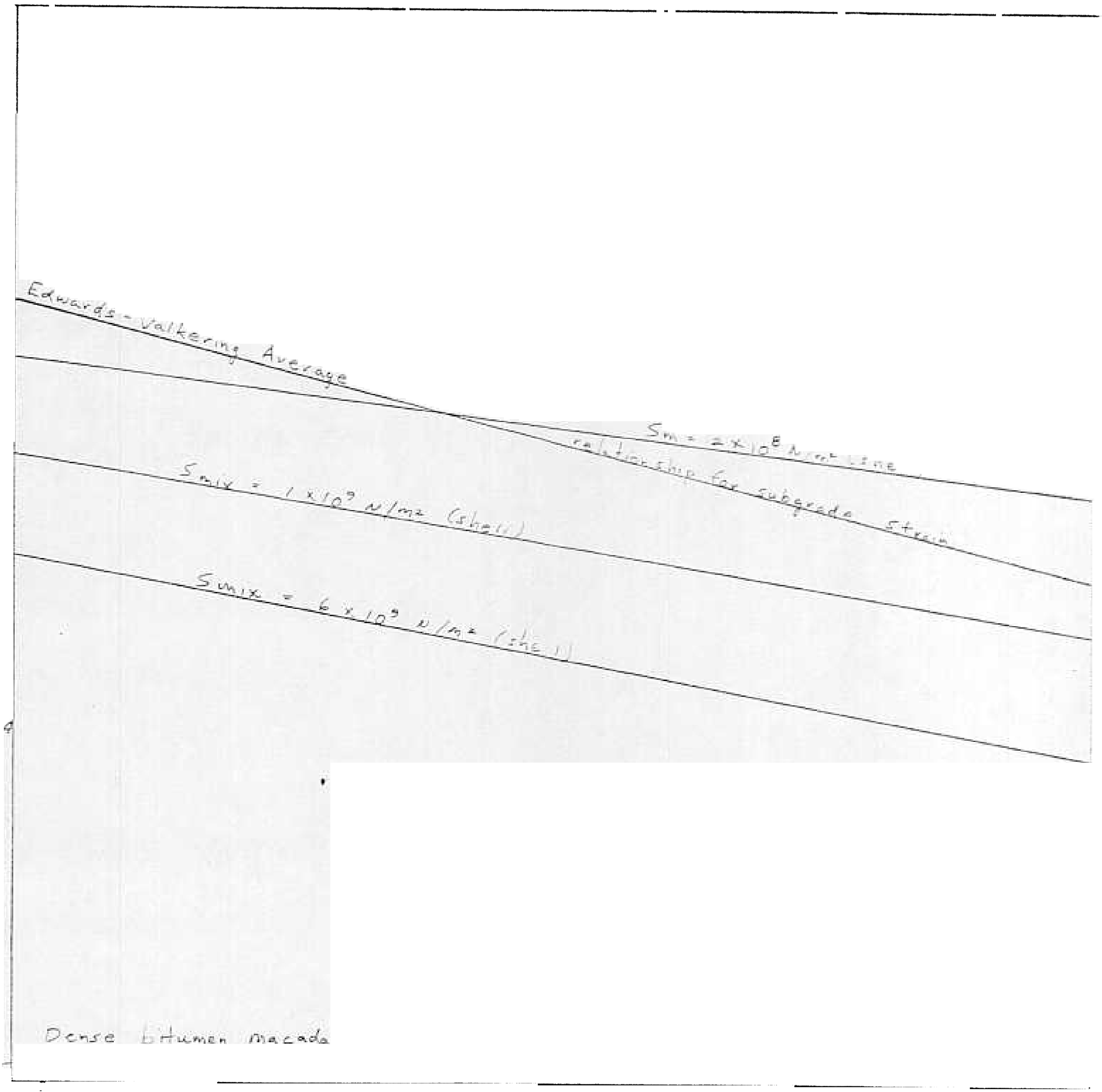
The Shell Asphalt Pavement Design Method (Ref. 13) is based on the theory of linear elasticity and provides a set of charts from which to assess the thickness requirements of the pavement.

The pavement structure is a linear elastic three-layered system in which materials are characterized by Young's modulus of elasticity and Poisson's ratio. The classical theoretical assumptions are used for all material characteristics. Two important considerations of the method are the effects of asphalt temperature which makes possible designs appropriate for various climates and the use of bituminous mixes with differing properties.

The bottom layer (subgrade) of the three-layer pavement structure is semi-infinite in the vertical direction. The middle layers represent the unbound base layers. The top layer represents the asphalt bound layer.

The BISAR computer program was developed on the basis of the three-layer model. It permits the calculations of stress, strains and displacements at any point in the multi-layered system. It also includes principa stresses and the direction in which they act. The maximum value of stresses and strains in the pavement structure are determined by the BISAR program for the compressive strain at the subgrade and the horizontal tensile strain in the asphalt bound layer. The compressive strain is thought to control permanent deformation of the subgrade and the tensile strain controls cracking of the asphalt layer. (Author's Note: The deformation in the subgrade of relatively thick pavements (30 inches) is probably minimal. Therefore, deformation is probably controlled by the base course and subbase layers. In cases where high modulus of elasticity ratios exist between base and asphalt layers, the maximum horizonta asphalt strain is found at a higher level in the asphalt layer.)

a. Fatigue Criteria. The fatigue criterion in the Shell Method are based on the permissible vertical compressive strain in the surface of the subgrade (which is probably the layer below the base in thick systems) and the permissible horizontal tensile strain on the bottom of the asphalt concrete layer as a function of the number of strain repetitions and modulus of the asphalt concrete. The permissible subgrade strain is a function of the number of 18 kip axle dual wheel load applications associated with a Present Serviceability Index (PSI) = 2.5. The average relationship for subgrade by Edwards and Valkering is: (used in the Shell Method)


Edwards-Valkering Average
Sm = 2 x 10^8 N/m² Line
relationship for subgrade strain
Smix = 1 x 10^9 N/m² (Shell)
Smix = 6 x 10^9 N/m² (Shell)
Dense bitumen macadam

$$\varepsilon_s = 2.8 \times 10^{-2} \times N^{-0.25} \quad \text{(Equation 12)}$$

Where:

ε_s = permissible compression strain in the subgrade

N = number of strain repetitions.

The above equation is graphically represented in Figure 11.

The asphalt concrete fatigue strain criterion was derived from laboratory tests of various asphalt mixes. The permissible asphalt strain was calculated as a function of the stiffness modulus of various mixes and the number of strain repetitions. Sample Shell curves are shown in Figure 12.

FIGURE 12

Permissible Asphalt Strain as a Function of Mix Stiffness Modulus. Dense Bitumen Macadam.

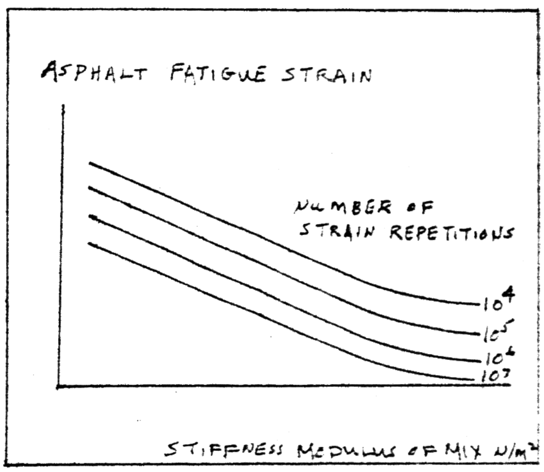


FIGURE 12

Fatigue is the phenomena of repetitive load-induced cracking due to a repeated stress or strain level below the ultimate strength of the material. Fatigue tests are conducted by several test methods and various specimen sizes. Obvious differences exist between fatigue tests and fatigue criteria. These differences present a significant variation when interpretation of the fatigue curves is made to determine the most critical fatigue period. However, when cumulative damage techniques are used with individual criterion, a significant practical difference is not found in the fina design thickness of asphalt concrete necessary for fatigue resistance.

In the Shel Method, the fatigue design curves are developed for acceptable strain levels and stiffness modulus of mix as a function of the number of strain repetitions to failure. Using the BISAR computer program and Burmeister three-layer stress-strain equations, fatigue curves are constructed for subgrade strain and the number of load applications to failure and for asphalt strain as a function of the stiffness modulus of the mix and the number of load applications to failure.

Relating the Shell fatigues curves to the experimental pavement necessitates calculating strain values with the Burmeister three-layer strain relationships. The stress-strain calculations require intensive and difficult solutions which can best be evaluated by computer analysis. However, the Boussinesq one-layer strain equations (Equation 3a and 3b) can be solved without complicated computer analysis. The vertical and horizontal Boussinesq strain equations were solved for the bituminous layers of the experimenta test sections. The calculated strains are shown in Table 14, and were determined by Equations 3a and 3b. The basic assumptions for the parameter value of the equations were determined as outlined in the following report sections. The parameter "E", (Modulus of elasticity of the bituminous layers) was assumed from Bituminous Mix Stiffness Modulus, Table 16 and the parameter "P" (contact pressure of the tire) was determined

TABLE 14

CALCULATED STRAIN VALUES

ONE LAYER EQUATIONS

$$\varepsilon_z = 1.5 \frac{P}{E_1} B, \quad \varepsilon_r = 1.5 \frac{P}{E_1} C$$

(from Witzack and Yoder: Page 29 and coefficients B&C from table 2.2, Page 30)

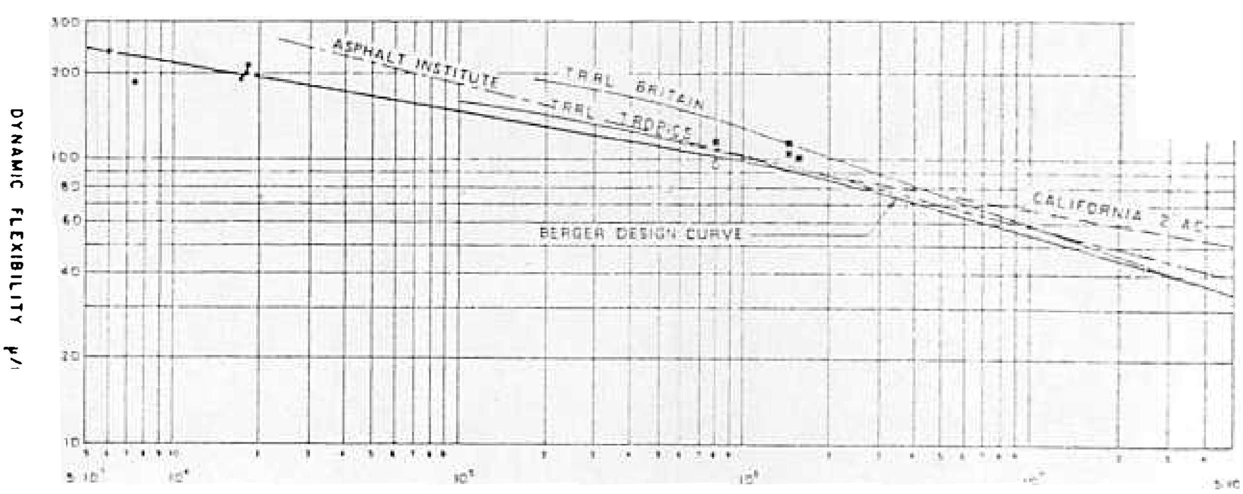
Bituminous Concrete Strain

Test Section	Vertical Strain ε_z	Horizontal Strain ε_r
1	3.45×10^{-5}	-1.72×10^{-5}
2	1.96×10^{-5}	-0.98×10^{-5}
3	2.42×10^{-5}	-1.21×10^{-5}
4	1.96×10^{-5}	-0.98×10^{-5}
5	3.45×10^{-5}	-1.72×10^{-5}
6	3.45×10^{-5}	-1.72×10^{-5}
7	2.84×10^{-5}	-1.42×10^{-5}
8	2.84×10^{-5}	-1.42×10^{-5}
9	2.84×10^{-5}	-1.42×10^{-5}

by measuring the tire contact area and dividing it by the total weight applied to the tire (Appendix C).

A comparison of the Shell strain curves, Figures 11 and 12, and the calculated strains values is difficult because the She curves are calculated by the three-layer equations and the actual properties of the Shell asphalt layers are unknown. However, the calculated strains in Table 14 indicate that strains in the experimenta test sections are low The low strain values correspond to the low deflection values shown in Table 5

Howkins (Ref. 15 and 16) presented a fatigue criteria method presumably developed by Berger. Although the development of this method cannot be substantiated in the iterature, Louis Berger, Inc. defined fatigue with the dynamic flexibility concept. The flexibility concept is a relationship between profilerometer deflection data and accumulated 18 kip loadings. The Berger flexibility curve is shown in Figure 13.



Fatigue Curves
Equivalent Standard 18 Kip Axles
(from Reference 16)
FIGURE 13

The dynamic flexibility equation by Howkins for the Benkelman beam is:

$$F = 55{,}900\ (^{d_0}/_{p})\ \text{microns/tonne} \qquad \text{(Equation 13)}$$

Where:

F = dynamic flexibility

d_0 = center deflection in units of 10^{-3}

p = force in lbs

Test section dynamic flexibilities and equivalent standard 18 kip axles are shown in Table 15 for the 80th percentile test section rebound data which was recommended by Howkins.

In Table 15 the predicted pavement life for the original Benkelman beam rebound data appears unreasonable for most test sections. The actual accumulated 18 kip loading (6.2 million) has exceeded the original predicted pavement life for Test Section 6E and 1W, and Test Sections 1E and 6W are near the actual loadings. However, the estimates for Test Section 3 and 7W are reasonable. The remaining pavement ife in accumulated 18 kip loads for the 1979 Benkelman beam rebound data also appears unreasonable for seven test sections but reasonable for the remaining nine test sections.

At this time, the use of dynamic flexibilities to predict pavement ife cannot be substantiated with the experimental pavement data. Further research and data are necessary for modification of the design flexibility curve with the experimental pavement data

TABLE 15

DYNAMIC FLEXIBILITIES
(Berger Design Curve)

	Pavement Life Predicted From Original Rebound Data			Pavement Life Predicted from 1979 Rebound Data		
Test Sections	Original Rebound	Dynamic Flexi-bility	Predicted Pavement Life	1979 Rebound	Dynamic Flexi-bility	Remaining Pavement Life
1E	9.8	61	6,500,000	15.5	96	1,200,000
2E	7.4	46	20,000,000	12.2	76	3,400,000
3E	8.2	51	12,000,000	14.3	89	2,300,000
4E	7.8	49	14,000,000	17.1	106	700,000
5E	8.1	50	13,000,000	12.4	77	3,400,000
6E	11.2	70	4,000,000	21.5	133	300,000
7E	7.5	46	26,000,000	9.4	59	9,500,000
8	7.9	49	14,000,000	17.4	107	680,000
1W	11.1	69	4,000,000	16.6	103	700,000
2W	5.5	34	50,000,000	11.4	71	4,000,000
3W	8.9	56	9,800,000	13.8	86	1,700,000
4W	6.0	38	38,000,000	18.8	116	360,000
5W	8.6	54	10,000,000	18.0	111	550,000
6W	9.7	61	6,500,000	18.7	116	360,000
7W	9.3	58	9,600,000	10.5	65	8,000,000
9W	8.0	50	13,000,000	13.7	85	1,700,000

b. <u>Bituminous Mix Stiffness odulus</u>. The stiffness modulus (Ref. 13) of an asphalt mix which varies considerably was found to depend solely on the bitumen content, volume of aggregate and the stiffness of the bitumen. The bitumen stiffness (asphalt modulus can be derived from the nomograph shown in Figure 14. The Shell equation (Ref. 13) for mix stiffness from which this nomograph was developed is as follows:

$$S_m = S_b \left[1 + \left(\frac{2.5}{n}\right) \left(\frac{C_Y}{1-C_Y}\right) \right]^n \qquad \text{(Equation 14)}$$

Where:

S_m = mix stiffness (Kg/cm^2)

S_b = bitumen stiffness (Kg/cm^2

$n = 0.83 \log_{10} (4 \times 10^5)/S_b$

$$C_Y = \frac{\text{Volume of Aggregate}}{\text{Volume of aggregate + Volume of asphalt}}$$

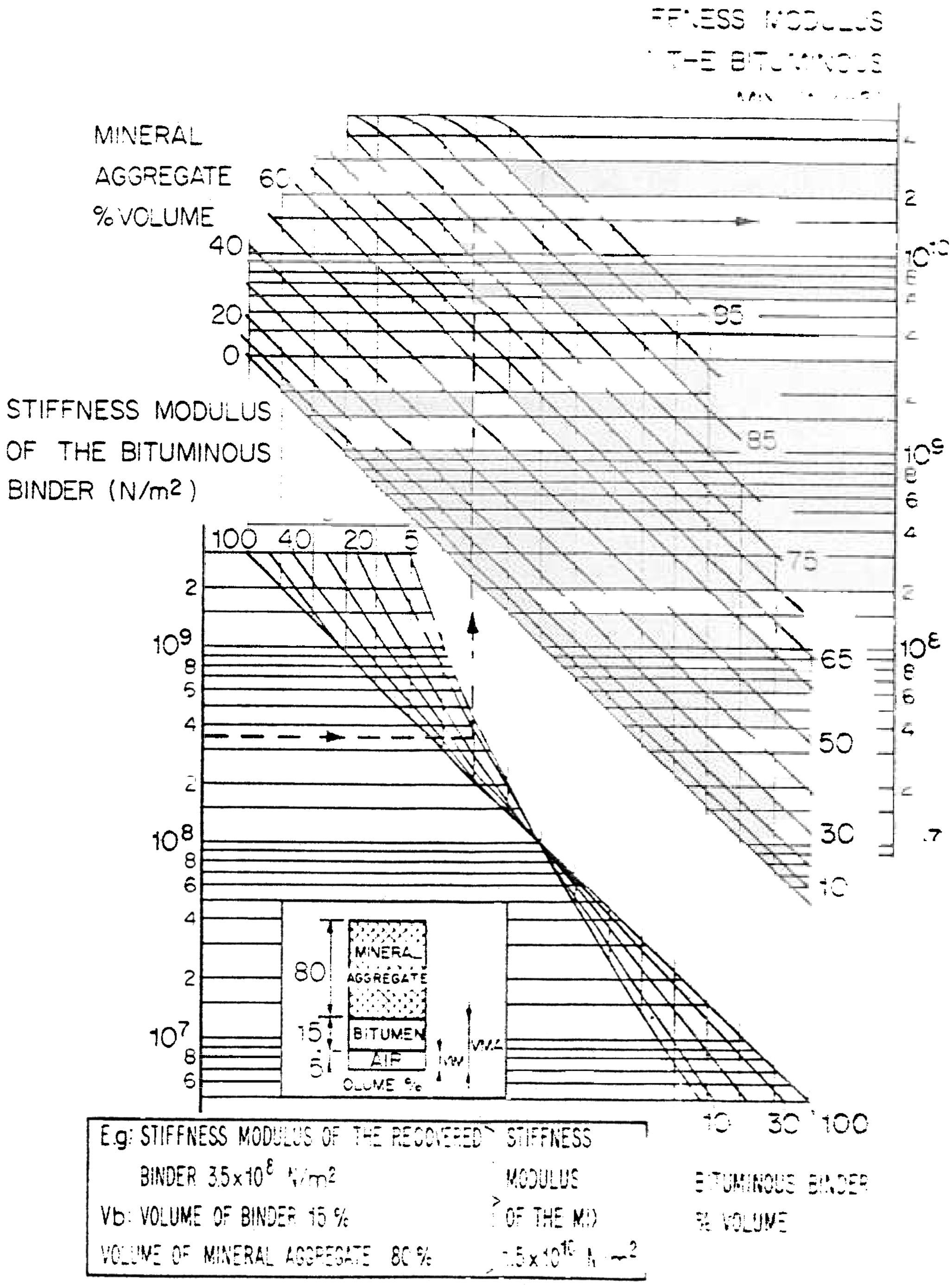
The bituminous mix stiffness modulus was calculated for the test sections and is shown in Table 16.

In addition to the above discussion, the bitumen (asphalt) modulus can be described as a function of the relevant temperature, loading times, and type of bitumen. A typical relationship between stiffness modulus of mix (which can be related to stiffness of bitumen) and temperature for specific asphalt mixes is shown in Figure 15.

Claussen (Ref. 13) substantiated the accuracy of the Shell nomograph (Figure 14) with extensive measurements on a large number of asphalt mixes. However, without laboratory tests of the asphalt mixes of the experimental pavement, the validity of the test section stiffness modulus of elasticity cannot be substantiated. Bituminous mix modulus of

FIGURE 14

(From Claessen, et al. - Ref. 13, Pg. 48)



Nomograph for predicting the stiffness modulus of bituminous mixes

TABLE 16

BITUMINOUS MIX STIFFNESS MODULUS

EXPERIMENTAL TEST SECTION BITUMINOUS MIXES

Bituminous Mix	Bituminous Stiffness (Kg/cm^2)
FABC-Top	5.08×10^4 Kg/cm^2
MABC-Top	5.9×10^4 Kg/cm^2
FABC-Bottom	7.0×10^4 kg/cm^2
Bit. Stab. Base (Gravel)	8.32×10^4 Kg/cm^2
Bit. Stab. Base (Stone)	8.32×10^4 Kg/cm^2

FIGURE 15

Stiffness Modulus Curves of Representative Asphalt Mixes as a Function of Temperature. (Reference 13)

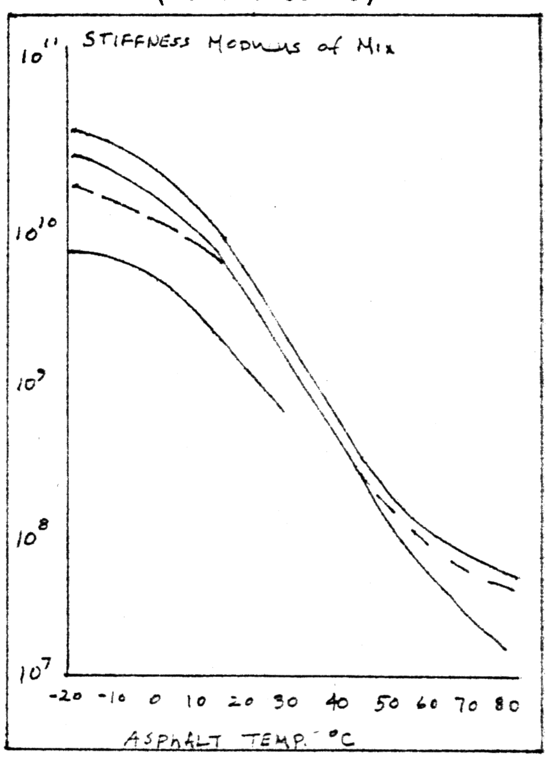


FIGURE 15

elasticity varies excessively between laboratory materials and field materials. Therefore, the bituminous mix stiffness modulus shown in Table 16 are estimates of an abstract property of the bituminous (asphalt) mix.

c. Thickness Design. Unfortunately, the Shell thickness design curves are not available in entirety for analysis of the experimental test sections. However, using design input procedures from the Shell Method and the two layer multi-elastic equations, design thickness estimates can be made that should reasonably simulate actual pavement thickness.

The effectiveness thickness can be determined by the equation:

$$t = \sqrt[3]{\frac{D\ 12(1-u^2)}{E}} \qquad \text{(Equation 6)}$$

which is discussed in a previous section. The flexual rigidity parameter "D" is defined by Poissons ratio, supporting layer modulus of elasticity and the characteristic length which is further defined by the ratio of the deflection basin measurements.

In the above expressions, assumptions for Poissons ratio can vary from 0.35 to 0.50 depending on the research source.

All parameters in the above equations are subject to variability and inherent errors from estimates of abstract properties. The modulus of elasticity of the supporting layer and the asphalt bound layer vary considerably from research source to source and should be substantiated by laboratory tests. The Hogg and Hertz equations were developed by Wiseman for flexible pavements from expressions for a rigid plate on a fluid foundation. The equations are essentially unsubstantiated by extensive actual field data. Therefore, the application of these equations to the experimental test sections is, indeed, experimental.

A sample calculation for effective thickness is presented here using the 1979 data from Test Section 1E (4" top course and binder course and 6" dry bound macadam base course).

EXAMPLE PROCEDURE (TEST SECTION 1E)

1979 Benkelman Beam Data

	20th Percentile rebound	Average Rebound	80th Percentile Rebound
d_r	= 0.0070 in.	0.0086 in.	0.0102 in
d_o	= 0.0131 in.	0.0143 in.	0.0155 in.

Howkins (Ref. 16) used 20th percentile "D" values which actually represents the worst "D" condition and the average "E_s" value. The 80th percentile rebound values are the compliment of the 20th percentile values and represent the worst rebound data. Effective thickness wi be calculated for averages and the 80th and 20th percentile data

The characteristic length "ℓ" is:

$$\ell = 56\ (1 - {}^{d_r}/_{d_o})^{-.46} - 50 \qquad \text{(Equation 8)}$$

Where:

ℓ_{20} = 29.3 cm

$\ell_{Avg.}$ = 35.4 cm

ℓ_{80} = 41.7 cm

The characteristic length is apparently a measure of the deflection bow shape. However, the characteristic length is not purely defined in the literature and its meaning is not entirely clear. Additional research is necessary to clarify the characteristic length

The pavement flexibility is:

$$F = 5.59 \times 10^5\ (d_o/P),\ p = 9000\ \text{lbs.} \qquad \text{(Equation 13)}$$

Where:

F_{20} = 81.4 microns/tonne

$F_{Avg.}$ = 88.8 microns/tonne

F_{80} = 96.3 microns/tonne

The flexibility, which was previously discussed, did not appear to adequately define the experimental pavement conditions. Therefore, its use in the claculation of the effective thickness is questionable

The pavement stiffness (Ref. 16) is the load required to cause a unit center deflection:

$$S = 10{,}000/F \qquad \text{(Equation 15)}$$

Where:

S_{20} = 122.9 tonne/cm

S_{Avg} = 112.6 tonne/cm

S_{80} = 103.8 tonne/cm

Howkins (Ref. 16) ratio of the measured pavement stiffness to the theoretical point load stiffness is:

$$S/_{So} = \frac{.287\ (\ell + 10)^{2.22}}{\ell^2} \qquad \text{(Equation 16)}$$

Where:

$(S/_{So})_{20}$ = 1.16

$(S/_{So})_{Avg.}$ = 1.09

$(S/_{So})_{80}$ = 1.05

The subgrade modulus (Ref. 16) (supporting material modulus - i.e., In the two layer theory, the material immediately below the bituminous bound layer is the supporting material) s given by:

$$E_s = 1000S/4.14\ell\ (S/_{So} \qquad \text{(Equation 17}$$

Where:

$$E_{s_{20}} = 873.4\ \text{Kg/cm}^2$$

$$E_{s_{Avg}} = 704.8\ \text{Kg/cm}^2$$

$$E_{s_{80}} = 572.4\ \text{Kg/cm}^2$$

(The estimated Burmeister two layer E_s value is 945 Kg/cm^2 for the 80th percentile deflection value. These calculations are shown in the Appendix It is anticipated that the different values calculated by the Burmeister equation and the Hertz-Hogg equation will be discussed in a later report.)

The rigidity of the bituminous pavement structure s represented by:

$$D = E_s l^3/1.5 \qquad \text{(Equation 7)}$$

Solving this equation, which relates rigidity to the characteristic length and support (subgrade) materia modulus, yields:

$$D_{20} = 1.46 \times 10^7 \text{Kg - cm}$$

$$D_{Avg} = 2.08 \times 10^7 \text{Kg - cm}$$

$$D_{80} = 2.77 \times 10^7 \text{Kg - cm}$$

Then, the effective thickness of the actua 4-inch bituminous layer is represented by:

$$t = \sqrt[3]{\frac{D\ 12(1-u^2)}{E}} \qquad \text{(Equation 6)}$$

where u = 0.5 and E = bituminous layer modulus of elasticity, which was previously calculated, as assumed to be the highest modulus value of all bituminous bound materials, i.e., $E = 7.0 \times 10^4 \text{Kg/cm}^2$.

Thus we have:

$$t_{20} = 12.3 \text{ cm}, 4.8 \text{ in.}$$

$$t_{Avg} = 13.8 \text{ cm}, 5.5 \text{ n.}$$

$$t_{80} = 15.2 \text{ cm}, 6.0 \text{ in.}$$

The inconsistency between the actual 4-inch thickness and calculated effectiveness thicknesses is attributed to inexperience with the equations and the selection of appropriate alues of dependent variables. The application of abstract values to an extrapolated equation, in this case, yields values that do not accurately define real 4-inch thick asphalt pavement. In actuality, the 4-inch bituminous top and binder course have not failed but exhibit, through rebound values, a decreased structural strength. However, the calculated effective thickness indicates that the pavement is essentially thicker than actuality.

An analysis of the effective thickness of the experimental test sections is shown in Table 17. The effective thickness was calculated with the average rebound values for the characteristic length which resulted in the average flexual rigidity value "D". In Table 17 four test sections (1E and 1W dry bound macadam base course and 5E and 5W penetration macadam base course) indicate a greater effective thickness than the actua thickness of the bituminous bound layer. At this time the phenomena cannot be explained adequately. The other effective thickness could be realistic. However, the discrepency with the above four test sections provides some skepticism as to whether this method can adequately describe the conditions of the experimental test sections.

EFFECTIVE THICKNESS

(Bituminous Bound Layer)

Average Values

Test Section	Actual Bit. Bound Layer Thickness* (in.)	Character-istic Length (cm)	Flexibility D ($X10^5$Kg-cm)	Effective Thickness (inches)
1E	4	35.4	208	5.5
2E	10	34.8	269	5.6
3E	8	40.6	357	6.1
4E	10	33.2	173	4.8
5E	4	30.1	188	5.3
6E	4	19.3	38	3.1
7E	6	24.7	148	4.6
8E	6	19.7	49	3.2
1W	4	27.6	124	4.6
2W	10	37.7	322	6.0
3W	8	31.8	189	5.0
4W	10	35.4	209	5.2
5W	4	28.4	112	4.4
6W	4	20.7	50	3.4
7W	6	33.4	296	5.8
9W	6	23.4	88	3.9

*Includes top and binder course and stabilized base course but does not include penetration macadam in Test Section 5E and 5W.

D. Discussion of Non-Destructive Testing Devices

Bush (Ref. 18) evaluated the Benkelman beam, Dynaflect, the Falling Weight Deflectometer, Road Rater Models 400, 510 and 2008 and the WES 16 kip Vibrator for deflection evaluation of light aircraft pavements. The report concluded that the standard Dynaflect, Road Rater 400 and Road Rater 510 had similar operating costs and had similar percent accuracy. The Road Rater Model 2008 rated overall slightly better than Models 400 and 510. The falling weight deflectometer indicated the best accuracy but the next to the highest operating costs after the Benkelman beam.

The report recommended three testing devices for the development of the light airport evaluation methodology: a) The Dynaflect, (b the Road Rater Model 2008 and (c) the Falling Weight Deflectometer.

Howkins (Ref. 16) demonstrated the Road Rater 510 in a dynamic pavement deflection survey for the Department. It was shown that the Road Rater correlated wel with the Benkelman beam and the Road Rater had the ability to measure more data points in equal time. However, no attempt was made to verify the application of the Road Rater data.

Majidzadeh (Ref. 19) developed pavement condition evaluation techniques and analysis procedures for flexible and rigid overlay pavement design using the Dynaflect and Road Rater data Claussen Ref. 14) developed a pavement evaluation method by which structural properties of the pavement can be derived from surface deflection and the shape of the deflection bowl. The Claussen study used the Falling Weight Deflectometer for the pavement evaluation and used the Shell Design Charts to determine a required overlay thickness. Both studies, ajidzade and Claussen, demonstrate the ability of non-destructive testing equipment to evaluate pavement conditions and overlay design.

The utilization of the dynamic testing equipment would greatly aid in the evaluation of the experimental test sections. An evaluation of the test sections by dynamic testing equipment would establish structural ratings for the determination of pavement failure criteria which would assist maintenance personnel in establishing structural overlay requirements

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APPENDICES

APPENDIX

VESYS STRUCTURAL PAVEMENT DESIGN METHOD

The VESYS (Ref. 19) mechanistic structural subsystem for pavement design was developed by the Office of Research, Federal Highway Administration for a broad range of paving materials, axle loads and environmental conditions. In this method, a given design is evaluated by determining its expected structural response to anticipated loading and environmental condition.

The VESYS computer program which is known as VESYS IIM predicts the performance of a pavement in terms of its present serviceability index (PSI) The PSI concept which was derived from the AASHO Road Test analysis, is a function of structural integrity, continuity and deformation of the pavement system. The computer program expresses distress in the form of cracking, rutting and roughness variables, using information on laboratory materials properties, geometry, traffic and environment. The damage indicators are used in a distress performance relationship to predict the serviceability history of the pavement.

The predictions (Ref. 21) of the VESYS pavement distress were compared by Rauhut, et al, to data from the Brampton and AASHO Road Tests. The comparisons show that the VESYS performance predictions can not be made at a significantly high confidence level. The inadequacy of the predictions is attributed to the inability of the present state-of-the-art of material characterization

APPENDIX E

KENTUCKY MULTILAYERED ELASTIC ANALYSIS

The Kentucky multilayered elastic pavement design method (Ref. 22, 23, 24, 25) was developed by the Kentucky Bureau of Highways from observations of pavement performance and empirical pavement deflection data. Correlation of empirical and theoretical deflections and strain and deflection relationships were obtained using the Chevron N-layered computer program furnished by the Chevron Research Company

The design principle (Ref. 22, 23 is based on limiting strain in the asphalt layers and in the underlying soil. In essence, the theoretical design is accomplished by investigating the maximum strain conditions at critical locations in the pavement interfaces. The pavement is designed to maintain strains within safe limits for the design life of the pavement.

The computed developed curves (Ref. 22, 24) of the Kentucky pavement design originated with input assumptions regarding the asphalt layer modulus of elasticity, Poissons ratio, and the Kentucky CBR values The input assumptions were based on Kentucky highway conditions. The imiting strain criterion of the asphalt concrete layers were based on interpretive analysis of pavement and fatigue test data from other sources

Verification of the Kentucky design system involves 40 years of Kentucky design and behaviora experience and a full depth asphalt concrete research pavement. The design system correlates well with the AASHO Road Test data. The Kentucky moduli solutions show excellent agreement with AASHO field data and theory.

APPENDIX C

DETERMINATION OF SUPPORT MODULUS FOR TEST SECTION 1-E

Burmeister 2-layer equation for deflection:

$$\Delta = 1.5 \frac{pa}{E_2} F_2$$

Where:

p = unit load on plate

a = radius of plate

E_2 = modulus of elasticity of lower layer

F_2 = dimensionless factor depending on the ratio of moduli of elasticity of the subgrade and pavement as well as the depth of radius ratio

Calculation for "p" unit load on dual tires:

Contact area of one tire = 74 sq. in

Tota contact area = 148 sq. in.

$$P = \frac{\text{Total axle load}}{\text{area}} = \frac{9000\#}{148 \text{ in}^2} =$$

$$P = 60.8 \text{ lbs./in}^2$$

2. By definition $a = \sqrt{\frac{P}{p\pi}}$ $\quad a = \sqrt{\frac{9000\#}{60.8\pi}}$ $\quad a = 6.8''$

3. The 80th percentile deflection = 0.0155

4. The Burmeister F_2 factor is calculated from Figure 27, Yoder and Witczak, where assumed E_2 = 8160 psi from Hertz-Hogg calculations and E_1 = 779,688 psi from Shell calculations for bitumen stiffness

$E_2/E_1 = 1/100$

$x/a = 0.6$

and $F_2 = 0.35$ from Figure 27.

Solve for E_2:

$$E_2 = 1.5 \left(\frac{pa}{\Delta}\right) F_2$$

$$E_2 = 1.5 \frac{(60.8)(6.8)}{0.0155} (.35)$$

$$E_2 = 13,500 \text{ psi}$$

$$E_2 = 945 \text{ Kg/cm}^2$$