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EVALUATION OF BITUMINOUS MIXES IN PAVEMENT STRUCTURES

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FINAL REPORT

By

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16. Abstract

A summary of the research methodology, results, conclusions and recommendations of a five-year study of bituminous mixtures used in Oklahoma flexible highway pavements is presented in this terminal report. The research project was concerned with three types of surface failures that occur in flexible pavement, i.e., decreased skid resistance, rutting and transverse cracking. The investigation was primarily directed toward evaluating and correlating the properties of asphalt-aggregate paving mixtures with these specific types of pavement distress.

Siliceous (polish resistant) aggregates had no detrimental effects on stability or cohesion of surface course mixtures prepared in the laboratory, but some aggregates had high stripping tendencies. Asphalt-bound pavement layers contributed to the development of surface ruts. Higher field densities and stabilities of the mixtures should be required and more stringent material specifications are needed. To alleviate transverse cracking, low-temperature behavior of asphalt cements and mixtures should be considered in the standard mix design procedures. The stiffness modulus concept is applicable to Oklahoma materials and conditions.

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Research Project 72-03-3
Joint Highway Research Program

Conducted for the
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Oklahoma State University
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PREFACE

This research project was concerned with asphalt paving mixtures used in Oklahoma and their relationship to certain types of surface failure conditions that occur on bituminous pavements within the state. The project involved laboratory studies of the properties and characteristics of the asphalt-aggregate materials and mixtures used in surface, leveling and base course layers of the pavement. Extensive field investigations, related to the performance of these paving mixtures under in-service highway conditions, were also made. Special instrumentation and procedures were developed to assist in the collection of data in both the field and laboratory portions of the study.

The results of this work substantiated the findings of other investigators, as well as some previously advanced ideas based on experience and knowledge of asphalt paving technology. The results and conclusions are significant in that they are based on the behavior of indigenous materials under existent conditions and show direct connections between the paving mixture properties and the respective surface distress conditions. Implementation of the recommendations relative to the materials used and the design and construction of the paving layers should result in improved pavement performance.

Many people have contributed their efforts and knowledge to this study. The assistance and valuable contributions of Larry S. Marr, Juan G. Wiegering, Miller C. Ford, Jr., Samuel Oteng-Seifah, Reynolds H. Toney, Magdy S. Nour El Din, and Ismail M. Basha are gratefully acknowledged. The Research and Development Division of the Oklahoma Department of Transportation should also be recognized for their excellent cooperation and assistance in the conduct of the project work.

Support for this project was provided by the State of Oklahoma, Department of Transportation, in cooperation with the U.S. Department of Transportation, Federal Highway Administration. This support is greatly appreciated.

P.G.M.

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CHAPTER I

INTRODUCTION

This is the terminal report of a five-year study of bituminous mixtures used in Oklahoma flexible highway pavements that was conducted by the O.S.U. School of Civil Engineering. The study was designated as Project 72-03-3 of the Oklahoma Department of Transportation and Oklahoma State University Joint Highway Research Program and was initiated in June, 1972. Support for this project was provided by the State of Oklahoma, Department of Transportation in cooperation with the U.S. Department of Transportation, Federal Highway Administration.

This research was concerned with three types of surface failure conditions that occur on flexible pavements, i.e., decreased skid resistance, rutting and transverse cracking. Since the design of a bituminous paving mixture, the properties of the mixture and its component materials, and the manufacture and placement of the mixture, greatly influence the ultimate behavior of the pavement under in-service conditions, the investigation was primarily directed toward evaluating and correlating the properties of asphalt-aggregate paving mixtures with these specific types of pavement distress.

The project was divided into three major areas of study. The principal objectives of the respective investigations were:

1. To evaluate the effect of siliceous (polish resistant) aggregates on the properties of standard asphalt concrete surface course

mixtures and to determine the stripping tendencies of these aggregates;

2. To develop instrumentation and procedures for accurately measuring rut depths on flexible pavements and investigate the contribution of the asphalt-bound pavement layers to rutting;

3. To determine the nature and causes of transverse cracks in flexible pavements and to relate certain properties of the paving mixtures to this type of failure.

During the study period, a series of five interim reports were prepared and submitted. These reports detailed the methods, equipment, and techniques employed for collecting and analyzing the data obtained in the field and laboratory phases of the respective studies and presented conclusions and recommendations drawn from the results. The purpose of this report is to review the project work accomplished and summarize the findings and recommendations resulting therefrom.

Although the studies were related, there was a certain amount of inherent diversity. Consequently, the major studies have been separated in the following chapters of this report. A brief discussion of the individual investigations is presented along with a condensation of the results obtained and a review of the more pertinent conclusions made. Significant recommendations based on the findings of all the studies are presented in the final chapter.

CHAPTER II

MIXES CONTAINING SILICEOUS AGGREGATES

Decreased skid or frictional resistance is a special type of flexible pavement distress that can be a definite safety hazard for the traveling public. While external factors such as surface water and tire condition contribute to skidding, the surface skid resistance is a direct function of the angularity, hardness, and resistance to polish of the aggregate used in the surface mixture. If the coarse aggregate used in the mixture has a proclivity to polish or become smooth under the abrasive action of traffic, the skid resistant qualities of the pavement surface decrease in direct proportion.

Many limestone aggregates used in Oklahoma flexible paving mixtures have a tendency to polish and over a period of time the decreased skid resistance of constructed pavements has become a serious problem. The Oklahoma Department of Transportation, as a result of a long-range investigation of skid resistance and the polishing tendencies of aggregates (1), has attempted to improve these conditions by permitting the incorporation of small quantities of siliceous aggregates in their standard surface course mixtures.

The polishing tendency of these silicious aggregates, referred to as "acid-insoluble" materials, is generally much lower than for carbonate aggregates. However, these materials have traditionally been considered detrimental to other desirable characteristics of the asphalt-aggregate mixture. This is primarily due to the relatively poor adherent properties of these siliceous materials with the asphalt binder. This portion of the investigation was devoted to a laboratory study and evaluation of the effects of siliceous materials on surface course mixture properties, such as stability, cohesion and stripping resistance, to determine if changes in aggregate specifications were warranted.

Materials Used

Asphalt Cement

The asphalt cement utilized in this research was produced from an Oklahoma crude using the steam and vacuum process of refining. The material was classified as an 85-100 penetration paving grade asphalt cement and was chosen to represent one of the more common binders used in asphalt pavement construction in Oklahoma. The physical properties of this asphalt cement are shown in Table I.

Aggregates

The sources of aggregate used in this study were selected by the Oklahoma Department of Transportation in accordance with their investigation of pavement skid resistance. A relatively pure limestone from Cooperton, Oklahoma, and an Arkansas River sand (Arkhola sand) from a source near Muskogee, Oklahoma, were used as a standard aggregate mixture. Other aggregates, primarily siliceous in nature, were selected for blending with the standard mixture in specified percentages. These aggregates were obtained from various sources presently furnishing large quantities of material for highway construction. A total of ten different sources were sampled and the aggregates included two

TABLE I

PROPERTIES OF ASPHALT CEMENT

Properties	ASTM Method	Test Value
Penetration, 77°F, 100 g, 5 sec	D 5	93
Ductility, 77°F, cm	D 113	150+
Viscosity at 275°F, Kinematic, cST	D 2170	400
Thin Film Oven Test	D 1754	
Penetration After Test, 77°F, 100 g, 5 sec	D 5	60
Percent of Original		64
Ductility After Test, 77°F, cm	D 113	150+
Average Weight Loss	D 1754	
Percent of Original		+0.018
Specific Gravity, 77°/77°F	D 70	1.003
Softening Point, °F	D 2398	118
Flash Point, °F	D 92	580+

types of limestone, three types of sandstone, one chert, and four types of gravel.

The aggregates are identified as to source location, geologic unit, geologic age (period) and general classification in Table II. The sample name corresponds to the city or town near the location of the pit or quarry from which the material was obtained. Table III lists some of the physical properties of these aggregates. The "Insoluble %" values in this table refer to the acid-insoluble residue percentage (IRP) of the aggregate as determined by Oklahoma Test Method OHD-L-25 (2).

Mix Design Procedures

The mix design procedures and specimen preparation techniques used in this study conformed to those used by the Oklahoma Department of Transportation. A more detailed discussion of these respective aspects is presented in Interim Report I (3). The aggregate gradation used in all mixes was that of the mid-point of the specification limits stipulated for an Oklahoma Type B surface or base course mixture (4). Mixtures containing 4, 4.5, 5, 5.5 , 6, and 6.5 percent by total weight were prepared for each aggregate or aggregate blend used in the study. Four test specimens were molded at each asphalt content for stability and cohesiometer tests. Additional specimens were molded for immersion-compression tests.

The Cooperton limestone-Arkhola sand combination was the "standard" aggregate mixture for the study. Results of previous studies (5, 6) have indicated that the coarse aggregate in the pavement surface governs, to a large extent, the skid resistance of the pavement. Therefore, coarse

TABLE II

AGGREGATE IDENTIFICATION AND SOURCE

OHD No.	Sample	County	Location[1]			Geologic Unit Period	General Classification
			Sec.	Twp.	Rg.		
38-01	Cooperton	Kiowa	32	6N	15W	Kindblade limestone Ordovician	Limestone
03-01	Stringtown	Atoka	16	1S	12E	Wapanucka limestone Pennsylvanian	Siliceous Limestone
08-01	Cyril	Caddo	36	6N	10W	Rush Springs Permian	Calcareous Sandstone
31-01	Keota	Haskell	23	10N	23E	Bluejacket Pennsylvanian	Siliceous Sandstone
46-01	Onapa	McIntosh	31	11N	17E	Bluejacket Pennsylvanian	Siliceous Sandstone
63-01	Asher	Pottawatomie	4	6N	4E	Wellington-Admire Permian	Chert Gravel
45-01	Broken Bow	McCurtain	4	7S	26E	Alluvial Deposit Quaternary	Siliceous Gravel
68-01	Gore	Sequoyah	19	12N	21E	Alluvial Deposit Quaternary	Siliceous Gravel
12-01	Hugo	Choctaw	36	5S	17E	Terrace Deposit Quaternary	Chert Gravel
58-01	Miami	Ottawa	31	29N	23E	Boone Mississippian	Chert

[1] Based on USPLS Indian Meridian

TABLE III

AGGREGATE PHYSICAL PROPERTIES

Sample	Bulk Specific Gravity	Absorption	L.A. Abrasion	Soundness $NaSO_4$	Soundness $MgSO_4$	Insoluble (%) (+No. 200 sieve)
Cooperton	2.67	0.8	24	0.8	4.4	1.2
Stringtown	2.57	0.5	22	4.4	6.3	72.8
Cyril	2.64	0.9	37	4.1	---	59.2
Keota	2.48	2.4	40	---	---	96.3
Onapa	2.47	4.1	35	8.9	---	92.1
Asher	2.46	3.2	25	6.5	---	99.8 chert
Broken Bow	2.69	1.3	25	---	---	98.3
Gore	2.68	0.6	29	---	2.7	97.9
Hugo	2.52	1.8	20	---	2.8	99.0 chert
Miami	2.56	1.2	23	2.9	---	95.4 chert

aggregate fractions of the various siliceous materials were incorporated in this standard mixture in amounts based on the acid-insoluble residue percentage (IRP) of each respective aggregate.

Aggregate combinations or blends containing 20, 30 and 40 percent (by weight of aggregate) acid-insoluble material in the coarse fractions, i.e., the fractions above the No. 10 sieve, were used for the mixtures. These percentages included the IRP contained in the Cooperton limestone. Sample calculations to determine the percentage of siliceous aggregate to be incorporated in a mixture are shown below.

Given: Onapa Sandstone IRP = 92.1 % a $\% = \frac{c-b}{a-b}$

Cooperton Limestone IRP = 1.2 % b

Find: % Onapa Sandstone (by weight of aggregate) for c
20 % acid-insoluble residue in mixture.

1. 20% - 1.2% = 18.8%

2. $\%\ \text{Onapa Sandstone} = \frac{18.8\%}{92.1\% - 1.2\%} = 20.68\%$

For the above example, 20.68 percent of the coarse fractions of the Cooperton limestone were replaced by like fractions of the Onapa sandstone to obtain 20 percent insolubles in the coarse aggregate portion of the mixture. Similar calculations were used for the 30 and 40 percent mixtures.

Test Procedures

Stability and Cohesiometer Tests

The ASTM standard method of test, D 1560, was used to determine the stability or resistance to deformation and the cohesion of the compacted asphalt-aggregate mixtures.

Immersion-Compression Test

An immersion-compression test was developed to determine the effect of water on the cohesion of the compacted mixtures. This test procedure was patterned after the standard ASTM method of test, D 1075. However, several variations were used to take advantage of available molding and test equipment. The specimens were molded with a motorized gyratory-shear compactor, rather than using the static double plunger compression method, and were tested for compressive strength using a Marshall stability testing head instead of in axial compression without lateral support. Also, a vacuum saturation process was used on the immersed specimens to insure maximum penetration of water into the densely compacted samples. The results of this "modified" immersion-compression test were considered indicative of the relative stripping tendencies of the siliceous aggregates used in the respective mixtures.

Static and Dynamic Stripping Tests

In order to obtain additional information on asphalt film retention by the siliceous aggregates in the presence of water, both static and dynamic immersion stripping tests were performed on asphalt coated samples of the respective aggregates.

Static Tests: The sample preparation and coating procedures followed the standard ASTM method of test, D 1664. After water immersion for 18 hours at 77°F (25°C), no stripping of any of the various aggregates was observed using the standard static test procedure. The test was made more rigorous by placing the containers of immersed samples in a 140°F (60°C) water bath for the 18 hour period. The amount of stripping was visually estimated according to the standard procedure. To partially eliminate the subjective aspects of this visual estimating method, comparision charts depicting samples with varying percentages of retained coating were prepared and used in evaluating the sample.

Dynamic Test: In order to accelerate the stripping action of water on the asphalt coated aggregate samples, a dynamic stripping procedure patterned after the method employed by Nicholson (7) was used. An apparatus was designed and constructed to hold six sample jars of approximately one quart (0.95 l) capacity. This apparatus rotated the jars about a horizontal axis at 40 rpm.

Preliminary tests indicated that a 4 hour period of tumbling or rotation on the apparatus at normal laboratory temperature was required to induce significant amounts of stripping of the coated samples. Visual estimates of the amount of stripping, i.e., percentage of retained coating, were made at the end of 1, 2, and 4 hours of tumbling. After completing this dynamic stripping test, the partially coated aggregate samples were used in the Surface Reaction Test to obtain a more quantitative measure of the amount of stripping that had occurred.

Surface Reaction Test

This test procedure was devised to measure the amount of exposed surface area on a "stripped" sample of asphalt coated aggregate by determining the gas pressure resulting from the reaction between a suitable reagent and the exposed aggregate surface. The reagents employed were hydrochloric acid, hydrofluoric acid, and a mixture of these two acids.

The test required precise measurement of the pressure generated when the aggregate sample was inundated by the acid. Since the temperature of such a reaction affects the volume of the gas, it was necessary to measure pressure and temperature simultaneously. A suitable reaction chamber or pressure vessel for this purpose was made by modifying a six quart (5.68 1) stainless steel pressure cooker. Fig. 1 shows the details of this pressure container. A pressure transducer and a thermistor were mounted on the lid of the container and connected to a dual-arm recording instrument. Pressure in the vessel could be determined to the nearest 0.025 psi (0.0017 kg/cm^2) and the temperature to the nearest 0.5°C. The pressure vessel, remote sensing therometer, and the recorder are described in detail in Interim Report I.

Calibration runs were made on uncoated samples of the respective aggregates cut to geometric shapes. These samples were carefully measured and the surface area calculated. The pressure-temperature curves plotted on the recorder chart for a stripped sample were analyzed and compared with the calibration curve for the uncoated aggregate sample. The change in gas pressure during a stated interval of reaction time was considered proportional to the exposed aggregate surface area. The procedure is explained in greater detail in Interim Report I (3) and in a paper by Ford and Manke (8). This method of test has been patented (9).

Specific Gravity Tests

While not an essential part of the primary objective of this study, several standard methods of determining the percent density of compacted asphalt-aggregate specimens were compared. The use of a proper specific gravity of aggregate is of paramount importance in the design of bituminous mixtures. In order to obtain the "actual" density of these mixtures, the specific gravity of the aggregate blend must be accurately

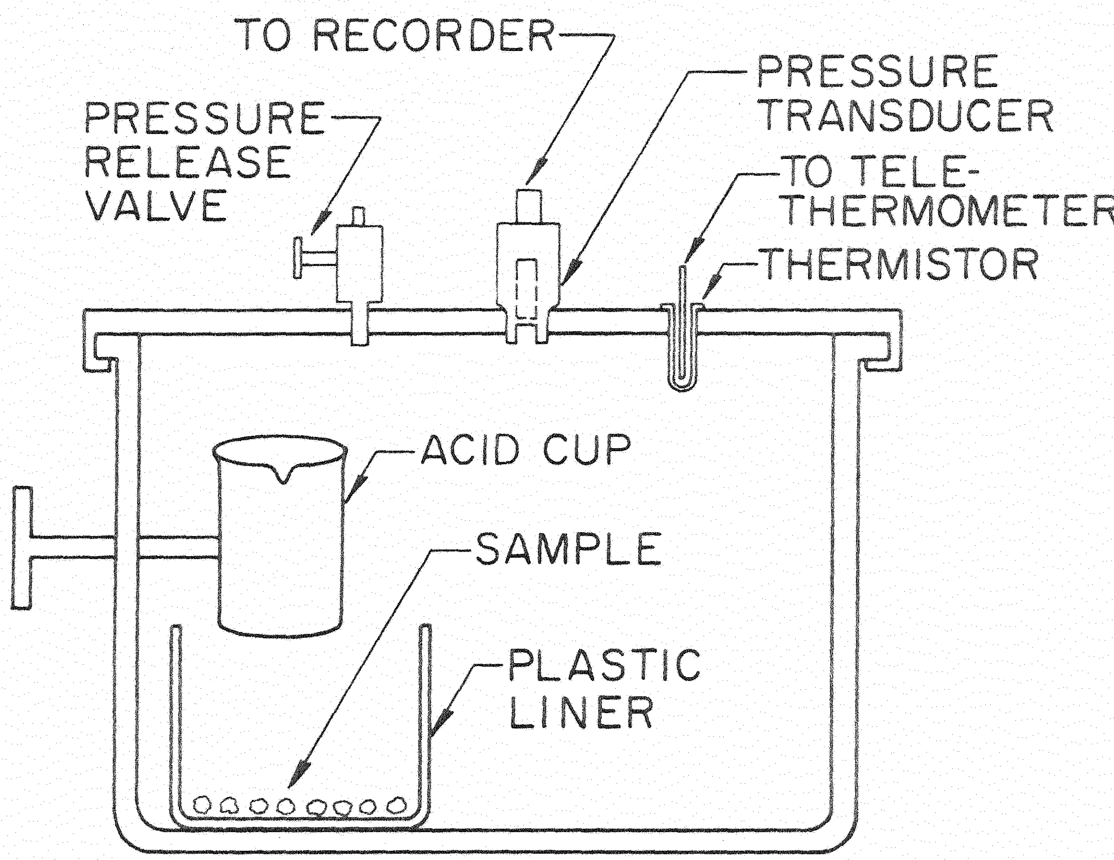

Figure 1. Details of Pressure Container Device

determined. Two of the methods used a calculative procedure in which the theoretical maximum specific gravity of the mixture was based on an average specific gravity of the combined aggregates. These combined aggregate specific gravities were obtained from the bulk specific gravities, ASTM standard methods of test, C 127 and C 128, and the bulk impregnated specific gravities, OHD-L-7 (2), of the aggregate blends used in the study. The third method was Rice's procedure for measuring the theoretical maximum specific gravity of bituminous paving mixtures, ASTM standard method of test, D 2041.

Results and Discussion

Stability and Cohesiometer Test Results

Table IV shows the stabilometer value, the cohesiometer value and the percent density (Rice's method) of the compacted specimens for each of the aggregate combinations at the selected optimum asphalt content. The selection of the exact mid-point gradation of the Type B specification limits resulted in VMA's ranging from 10.8 percent to 13.6 percent in the mixtures. Stability values were satisfactory but had to be obtained at optimum asphalt contents of 4 to 4.5 percent asphalt by weight of total mix. Some of these aggregate blends exhibited critical tendencies, and it would have been desirable to have increased the coarse aggregate fraction of the mixtures.

Generally, the stability curves for a given type of aggregate blend were similar and the stability values comparable. From the stability standpoint, no disadvantage could be assigned to the use of specific percentages of siliceous material in the standard mixture. With proper

TABLE IV

STABILOMETER VALUE, COHESIOMETER VALUE AND PERCENT DENSITY OF COMPACTED SPECIMEN AT OPTIMUM ASPHALT CONTENT

Aggregate	Insoluble Residue Percentage	Optimum Asphalt Content	Stabilometer Value at Optimum	Cohesiometer Value at Optimum	Percent Density of Compacted Specimen (Rice's Method)
Cooperton Limestone	na	4.25	42	192	96.5
Asher Chert Gravel	20	4.25	42	183	96.3
	30	4.25	38	187	96.2
	40	4.25	37	164	96.2
Miami Chert	20	4.50	39	163	96.5
	30	4.50	40	167	97.0
	40	4.50	38	185	96.7
Onapa Sandstone	20	4.50	42	200	96.5
	30	4.50	41	180	95.7
	40	4.50	43	156	96.1
Stringtown Limestone	20	4.25	42	177	96.3
	30	4.25	41	162	96.3
	40	4.25	40	175	96.1
Cyril Sandstone	20	4.25	39	196	96.4
	30	4.25	40	266	96.7
	40	4.25	38	204	96.6
Broken Bow Gravel	20	4.00	38	190	96.5
	30	4.00	35	307	98.2
	40	4.00	40	296	97.5
Gore Gravel	20	4.00	40	311	97.6
	30	4.00	36	302	97.0
	40	4.00	41	323	97.7
Hugo Chert Gravel	20	4.00	38	330	97.7
	30	4.00	36	278	97.9
	40	4.00	37	301	97.8
Keota Sandstone	20	4.00	44	275	97.0
	30	4.00	44	343	96.6
	40	4.00	42	394	97.6

adjustment of the mix gradation, it would be possible to achieve adequate stabilities at higher asphalt contents of all mixtures containing up to 40 percent of the various siliceous aggregates.

The tensile strength or cohesive resistance of a compacted asphalt-aggregate mixture is predominantly influenced by the inherent cohesive properties of the binder with adhesive forces developed at the asphalt-aggregate interfaces contributing to a minor extent. The results of the cohesiometer tests failed to indicate any significant effects due to the presence of the siliceous aggregates in the mixtures. In all cases, the values were well above the recommended minimum cohesiometer value of 50.

The cohesiometer test proved to be extremely sensitive to operator techniques. As indicated in Table IV, the magnitude of the test results increased from top to bottom or toward the latter part of the study. This was attributed not so much to actual increase in cohesion of the aggregate blends as to operator experience and the more uniform manner in which the tests were performed.

Immersion-Compression Test Results

No defininte trends with regard to the relative stripping resistance of the aggregate blends used in the stability test mixtures could be established from the results of the immersion-compression tests. Some indication of the durability of the mixtures containing large percentages of the siliceous aggregates was desired and so additional immersion-compression specimens were molded using the ten aggregate types. The coarse aggregate portion (plus No. 10 material) of these mixtures was composed entirely of the respective aggregate to be evaluated. For each of the aggregates, four specimens were molded at 4 percent and at 5 percent asphalt content. Table V shows the physical properties of these specimens and the results of the modified immersion-compression tests. Based on percent retained strength data, the relative stripping resistance of the aggregates (from excellent to poor) was: 1) Cooperton limestone, 2) Hugo chert gravel, 3) Asher chert gravel, 4) Stringtown siliceous limestone, 5) Broken Bow siliceous gravel, 6) Miami chert, 7) Cyril calcareous sandstone, 8) Keota siliceous sandstone, 9) Gore siliceous gravel, and 10) Onapa siliceous sandstone.

Film Stripping Test Results

The film stripping resistance of the various siliceous aggregates were evaluated by the static immersion and dynamic immersion tests. The results of the static immersion tests, at 77°F (25°C) and 140°F (60°C), on the respective coated aggregate samples are listed in Table VI. The results for each aggregate sample after 1, 2, and 4 hours of dynamic immersion testing are also presented in Table VI. In all cases, these values are based on visual estimation of the percent retained asphalt coating of the sample.

The results of the surface reaction tests are shown in Table VII. These are the quantitative values of percent retained coating determined on the same samples subjected to the dynamic immersion stripping test. Table VII is also a summary of the immersion-compression, film stripping and surface reaction test results, which were used to develop a relative ranking of the respective aggregates. The 4 percent asphalt content immersion-compression strengths were adjusted to obtain a relative maximum retained strength of 100 percent. That is, the 4 percent asphalt content values in Table V were divided by 1.14.

TABLE V

PHYSICAL PROPERTIES AND IMMERSION-COMPRESSION TEST RESULTS FOR 100% COARSE SILICEOUS AGGREGATE MIXTURES MOLDED BY MODIFIED PROCEDURE

Sample	AC (%)	Water Absorbed % (vacuum sat.)	Bulk Specific Gravity	Dry Str. (psi)	Retained Strength (%)
Cooperton	4	3.1	2.326	203	114
	5	1.5	2.375	214	116
Stringtown	4	2.3	2.249	246	95
	5	2.2	2.289	248	102
Cyril	4	3.5	2.298	251	88
	5	1.7	2.346	248	106
Keota	4	2.4	2.192	226	80
	5	2.3	2.248	236	97
Onapa	4	5.6	2.144	250	66
	5	2.7	2.194	290	76
Asher	4	3.6	2.189	249	95
	5	1.7	2.237	252	104
Broken Bow	4	3.1	2.286	233	94
	5	1.4	2.337	238	106
Gore	4	2.3	2.238	250	80
	5	1.5	2.282	254	96
Hugo	4	2.2	2.266	182	99
	5	2.2	2.324	186	105
Miami	4	3.0	2.244	235	89
	5	2.2	2.290	241	101

TABLE VI

RESULTS OF STATIC IMMERSION AND DYNAMIC IMMERSION STRIPPING TESTS

Aggregate	Static Immersion Ret. Coating (%)		Dynamic Immersion Ret. Coating (%)		
	77°F	140°F	1 hr.	2 hr.	4 hr.
Cooperton	100	85	95	90	85
Stringtown	100	65	95	90	85
Cyril	100	60	90	80	75
Keota	100	50	95	90	80
Onapa	100	50	95	90	85
Asher	100	90	95	90	80
Broken Bow	100	90	95	90	70
Gore	100	40	90	85	65
Hugo	100	95	95	90	80
Miami	100	70	95	85	75

TABLE VII

SUMMARY OF STRIPPING TEST RESULTS AND AGGREGATE RANKING

Aggregate	Relative[1] Immersion-Compression 4% AC	Static Immersion 18 hr. @ 140°F	Surface Reaction	Average	Rank
	% Ret. Str.	% Ret. Ct.	% Ret. Ct.	%	
Cooperton	100	85	90	91.7	1
Stringtown	83	65	93	80.3	4
Cyril	77	60	64	67.0	7
Keota	70	50	56	58.7	8
Onapa	58	50	68	58.7	9
Asher	83	90	74	82.3	3
Broken Bow	82	90	54	75.3	5
Gore	70	40	65	58.3	10
Hugo	87	95	78	86.7	2
Miami	78	70	60	69.3	6

[1] Relative immersion-compression values based on Cooperton limestone having a retained strength of 100 percent (Table V).

It was realized that these three tests were independent measures of stripping. While their results are not comparable with regard to units, they do provide some insight as to the relative stripping tendencies of the respective aggregates. The indicated ranking is based on an average of three test values for each aggregate. Classified as to type, the stripping resistance of the "limestones" was better than that of the "gravels" which, in turn, were better than the "sandstones."

Specific Gravity Test Results

Results of the bulk impregnated and bulk specific gravity tests on the various aggregates and blends are presented in Table VIII and Table IX. Because the bulk impregnated specific gravity test attempts to account for the variable absorptiveness of the aggregate for water and asphalt, the test value is a type of "effective" specific gravity. Thus, values obtained from this test should always be greater than those from the standard bulk specific gravity test. However, with only a few exceptions, the tabulated results show the bulk impregnated gravities to be smaller than the bulk specific gravities. This indicates consistent but erroneous results from one or both of these specific gravity tests. This discrepancy points out an innaccuracy inherent to percent density determinations when a "calculated" maximum specific gravity of the mix is used.

The percent density of a compacted asphalt-aggregate mixture varies inversely with the maximum specific gravity of the mixture. Any errors in the percentages of the respective materials and the specific gravities of the asphalt and the combined aggregate are carried over and reflected in the percent density of the mixtures. In this study, many of the

TABLE VIII

BULK IMPREGNATED SPECIFIC GRAVITY FOR BLENDED AGGREGATE MIXTURES

Aggregate	Bulk Impregnated Specific Gravity		
Cooperton Limestone (Standard Mix)	2.69		
Standard Mix Plus Siliceous Aggregate	Acid-Insoluble Residue 20%	30%	40%
Asher Chert Gravel	2.63	2.61	2.59
Miami Chert	2.65	2.61	2.59
Onapa Sandstone	2.58	2.58	2.53
Stringtown Limestone	2.59	2.56	2.54
Cyril Sandstone	2.61	2.57	2.56
Broken Bow Gravel	2.61	2.60	2.58
Gore Gravel	2.56	2.52	2.49
Hugo Chert Gravel	2.58	2.53	2.49
Keota Sandstone	2.54	2.51	2.49

TABLE IX

AVERAGE BULK SPECIFIC GRAVITIES

	Average Bulk Specific Gravity of Aggregate	Average Bulk Specific Gravity of Blended Aggregates		
Cooperton Limestone	2.67	2.66		
Arkhola Sand	2.65			
		Acid-Insoluble Residue		
		20%	30%	40%
Asher Chert Gravel	2.38	2.63	2.62	2.59
Miami Chert	2.53	2.65	2.64	2.63
Onapa Sandstone	2.33	2.62	2.59	2.57
Stringtown Limestone	2.52	2.64	2.63	2.61
Cyril Sandstone	2.63	2.66	2.65	2.65
Broken Bow Gravel	2.53	2.65	2.64	2.63
Hugo Chert Gravel	2.53	2.65	2.64	2.63
Gore Gravel	2.46	2.64	2.62	2.61
Keota Sandstone	2.37	2.62	2.61	2.59

percent density values for the compacted mixtures, obtained using bulk and bulk impregnated specific gravities of the combined aggregates, approached and some exceeded 100 percent. Such values are entirely unsatisfactory for mix design purposes.

On the other hand, basing percent density values on the "measured" maximum specific gravity of the mixture seems to eliminate the inaccuracies that can be ascribed to the use of a "calculated" maximum specific gravity. At least the values for the mixtures shown in Table IV were more realistic and acceptable. This procedure is straight-forward and theoretically sound since the mixture tested in truly representative of the actual components in the compacted specimens.

Conclusions

Based on the results of this portion of the overall investigation, the following conclusions can be made:

1. In order to improve the skid resistant qualities of the standard surface course mixtures used in Oklahoma asphalt pavements, various percentages of siliceous aggregates, i.e., acid-insoluble materials, can be used. Up to 40 percent of the coarse aggregate fraction of the mix can be replaced with like amounts of these materials with little or no detrimental effect on the stability of the mix.

2. The siliceous aggregates had no apparent effect on the cohesive resistance or tensile strength of the respective mixes, as measured by the cohesiometer test. In all cases, the cohesiometer test values were well above the recommended minimum value for such mixes.

3. The mid-point gradation of the Type B surface mix specifications was not ideal for the purposes of this study. The gradation should have

been coarsened to achieve a higher VMA value and permit the use of higher asphalt contents.

4. The modified immersion-compression test developed in this study can be used to evaluate the loss of cohesion resulting from water action on compacted asphalt-aggregate mixtures conforming to Oklahoma surface course specifications. In the case of mixtures containing siliceous aggregates, the results can be indicative of the stripping tendency and subsequent durability to be expected from such mixes.

5. The surface reaction test provides a quantitative measure of exposed surface on a "stripped" asphalt coated aggregate sample. This test, used in conjuction with a dynamic stripping procedure and/or the static immersion stripping test at 140°F (60°C), can also be used to determine the relative stripping tendencies of surface course aggregates.

6. Of the three methods employed to determine the maximum theoretical specific gravity of the asphalt-aggregate mixtures, the ASTM method of test, D 2041, resulted in more realistic and acceptable values of percent density for the compacted specimens.

CHAPTER III

RUTTING IN FLEXIBLE PAVEMENTS

Under applied wheel loads, bituminous pavements undergo both elastic and plastic deformations (10, 11). The elastic deformations are recoverable upon removal of the load but the plastic deformations remain permanent. These latter deformations are cummulative and depend on the magnitude and duration of loading and the temperature and stress history of the material layers (12, 13, 14). The vertical deformations, i.e., channels or ruts that form in the wheelpaths on flexible pavement surfaces are a performance problem on many Oklahoma highways.

Field and laboratory observations have indicated three major contributory causes of rutting. Regarding the asphalt bound layers of the pavement, these causes are:

1. post-construction differential densification of one or more of the pavement layers,

2. shear failure or lateral displacement of material from beneath the wheelpaths in one or more of the layers,

3. surface wear or erosion of surface material due to the abrasive action of traffic.

In addition, consolidation and/or shear failures in the non-asphalt bound base and subgrade materials can influence the total amount of surface rutting. Each of these factors may act singularly or in various combinations in a specific case.

The primary objective of this portion of the research project was to investigate rutting or channeling on high-quality flexible pavements in Oklahoma and to detect, where possible, evidence of the contribution made by the bituminous bound paving materials to this type of surface failure. The research did not deal directly with the influence or contributions to rutting of the subgrade soils and the non-asphalt bound base materials.

An apparatus was developed to accurately plot the profile of the pavement surface perpendicular to the center line. Rut depths were scaled directly from these profile traces and humping or heaving of the surface adjacent to the ruts was considered an indication of lateral creep of material (in one or more of the layers) from beneath the wheelpaths. Four inch (10.16 cm) diameter cores were taken at selected points along the transverse profile line. Percent density values of subdivisions of these core samples were determined and compared. Significant differences in these values for core material from wheelpath and non-wheelpath locations were considered due to differential densification by traffic. Surface nuclear densities were also obtained at these core locations and an attempt was made to correlate the respective density values. Stereo-photography was employed to obtain quantitative estimates of differential wear in the wheelpaths.

Transverse Profile Gage

The profile gage was developed to provide a portable and accurate means of obtaining continuous transverse profile traces of the surface of a traffic lane. In addition to the measurement of rut depths and surface heaves adjacent to the wheelpaths, these plotted profile traces or graphs provided permanent records of the test site conditions at a specific time in the service life of the pavement. Thus, the profile gage could be used to follow the development of rutting (15, 16) and employed to check transverse profile tolerances following construction.

The transverse profile gage consisted of a supported straight edge that would span a 12.0 ft (3.66 m) traffic lane, a trolley system and a recording system. The straight edge component served as a guide rail and a datum plane for the trolley system. The trolley traversed the straight edge on nylon rail track wheels and consisted of a 5.0 in. (12.70 cm) diameter teflon actuating wheel connected to both linear and helical potentiometers. The potentiometers were connected to an X-Y recorder such that the helical potentiometer would scale the transverse displacement and the linear potentiometer the vertical displacement of the actuating wheel as it traversed the surface of the pavement. Fig. 2 shows the device being used on the roadway. Details of the construction and operation of the transverse profile gage are presented in Interim Report II (17).

Field Test Procedures

Site Selection and Layout

Sixteen test sites were selected on two interstate highway systems (I-35 and I-40) in Oklahoma. Surface deformations at four test sites on flexible pavement sections constructed on each of the following types of base course materials were studied: 1) hot-mix-sand-asphalt (HMSA), 2) soil cement base (SCB), 3) black base (BB), and 4) stabilized aggregate base course (SABC). These are the common types of base materials specificed by the Oklahoma Department of Transportation.

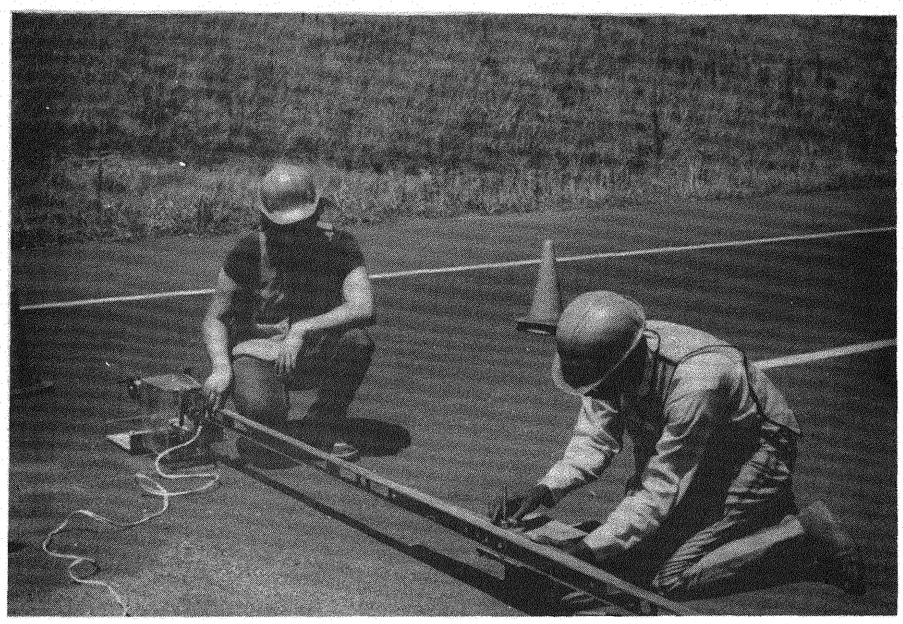

Figure 2. Straight Edge and Trolley Unit

It should be pointed out that the field study of these test sites was made after rutting had occurred. The approximate ages of the pavement sections at the time of the study ranged from 36 to 169 months. For this reason, exact amounts of surface deformation and densification in the pavement layers could not be determined. Measurement of rut depths and other deformations was based on a hypothetical datum or transverse profile at zero age, i.e., an assumed transverse profile at the time the pavement was opened to traffic.

Fig. 3 shows the layout of a typical test site on an interstate highway section. Line ABC represents the path traversed by the trolley unit of the profile gage. This line on the pavement surface was marked by a chalk line and pop bottle caps were nailed to the surface at the points indicated. Subsequent profile traces at these locations were originally planned and these bottle caps were used to preserve the exact location of the transverse profile line. The numbered points indicate the approximate locations where stereo-photos of the surface, nuclear density measurements, and pavement cores were obtained. As indicated, these points were located on an offset line, parallel to and 12.0 in. (30.48 cm) from the profile line in the direction of traffic.

In addition to the interstate test sites, two sites on a newly constructed segment of State Highway 51, northeast of Oilton, Oklahoma, were studied (16). The pavement sections at these locations consisted of a 9.0 in. (22.86 cm) thickness of HMSA base, 3.0 in. (7.62 cm) of Oklahoma Type A binder mix and 1.5 in. (3.81 cm) of Type C surfacing mix (4) on natural sugrade materials. Transverse profile tracings, stereophotographs, nuclear densities and core samples were obtained at these sites prior to opening the section to traffic. Similar data was obtained at these sites periodically over a two year period.

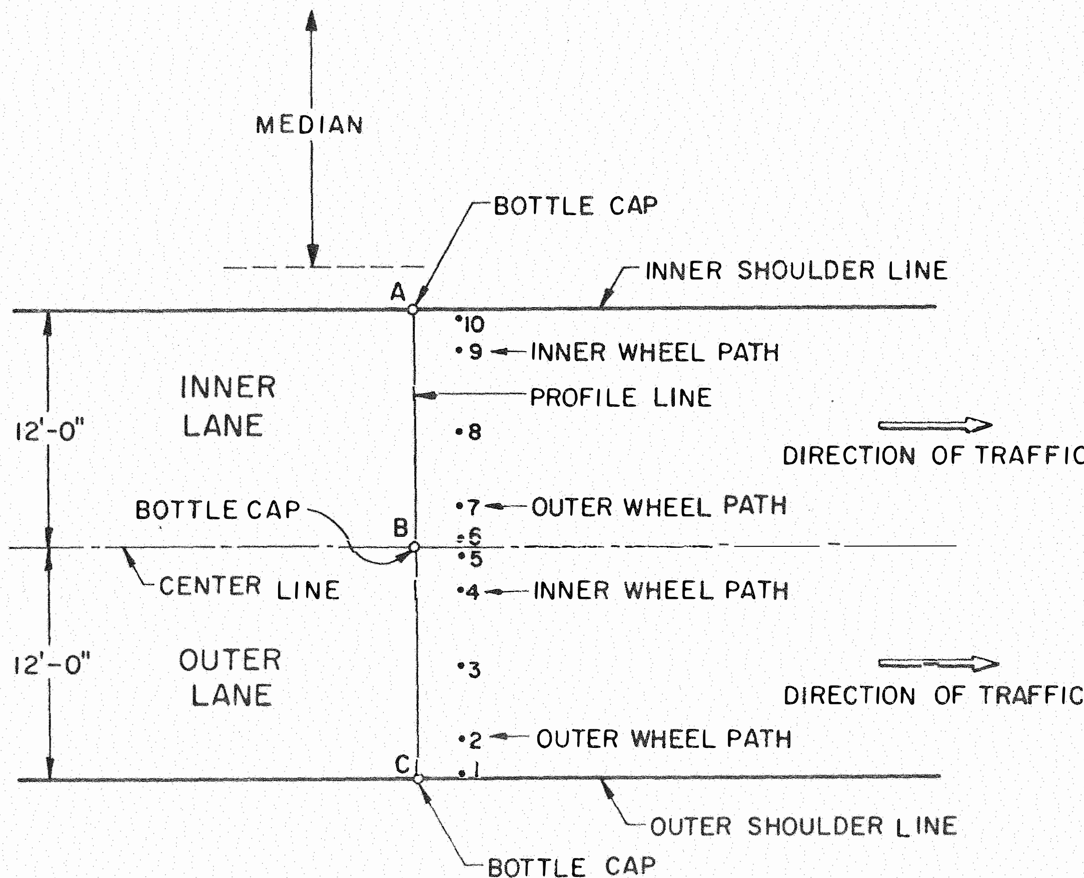

Figure 3. Layout of Transverse Test Point Locations at a Test Site

Profile Tracing

The straight edge of the profile apparatus was aligned directly over the transverse chalk line in a given lane. The height of the end supports of the rail were adjusted so that the linear potentiometer shaft on the trolley unit was positioned at the midpoint of its effective travel distance at each end of the rail. This assured that the ends of the straight edge were the same height above the pavement surface and provided the datum for the potentiometer so that both positive (subsidence) and negative (upheaval) deformations of the surface could be measured.

The necessary power for the potentiometers and the recording equipment was obtained from a standard 12 volt electrical system in a pickup truck that was used to transport the equipment to the field. An inverter supplied the necessary alternating current to the recorder and the potentiometers used direct current from the battery. After making the appropriate input and output power connections, the trolley unit was positioned at one end of the straight edge and the helical potentiometer set to its zero position. The trolley unit, with its actuating wheel in contact with the pavement surface, was then moved to the other end of the straight edge. The resulting linear and vertical displacements of the actuating wheel were recorded at a preselected scale on graph paper by the X-Y recorder.

Profile Measurements

The recorder pen trace on the graph paper was a continuous plot to scale of the transverse profile of the pavement surface in a traffic lane. The top surface of the straight edge was set parallel to an imaginary line joining the bottle cap points at the edges of the lane and this line was considered the datum for the profile measurements. Rut depth was measured as the maximum vertical displacement of the wheelpath surface from a straight line whose ends were tangent to the profile curve at adjacent points of maximum elevation. This corresponded to the common field measurement procedure using a scale and straight edge to determine maximum rut depth. Depending on the recorder scale used for the profile tracing, depths could be easily scaled to the nearest 0.01 in. (0.025 cm) from the transverse profile.

Upheaval of the pavement surface outside the wheelpath locations was considered an indication of lateral creep of material from beneath the wheelpaths. Since an "original" transverse profile of the pavement surfaces was not available, a straight line joining the end support points of the straight edge was assumed to be the original surface. Design records of the pavements at the study sites indicated that the sections were designed to have a uniform cross-slope for the lanes in a given traffic direction. Based on the records and the assumption, maximum vertical displacements of the surface above such a base line were scaled directly from the profile tracings.

Stereo-Photography

An offset line, parallel to and 12.0 in. (30.48 cm) from the profile line in the direction of traffic, was marked on the pavement. The selected test points (Fig. 3) on the profile line were laterally transferred to this offset line. Thirty-five millimeter stereo-photographs of the surface were then taken at these points using a stereo-photo box described by Schonfeld (18).

Nuclear Density Measurements

After taking stereo-photos of the pavement surface at each of the selected transverse test points, density measurements were obtained at these same points with a Troxler 2400 Surface Density/Moisture Gage. The backscatter method of measurement was employed and the test procedure followed those stipulated in the manufacturer's instruction manual (19) and ASTM method of test, D 2950.

Core Drill Operations

Full thickness cores, ranging from 6.0 in. (15.24 cm) to 14.0 in. (35.56 cm) in depth, of the asphalt bound pavement materials were cut at the ten selected test points along the previously described offset line. The 4.0 in. (10.16 cm) diameter cores were cut at the same points where the nuclear density measurements were obtained. Each core specimen was identified as to test site location and test point, wrapped in plastic bags and stored in insulated containers for transport to the laboratory.

Surface Condition Rating

In addition to the mechanical operations carried out at each test site, a subjective evaluation of the condition of the pavement surface was made. This was done to establish, in terms of a suitable qualitative index, the overall condition of the surface at these locations. The surface condition rating format suggested by Winnitoy (20) was modified to include the following conditions:

1. General structural condition.
2. Degree of surface weathering.
3. Uniformity of surface coloring.
4. Crack condition.
5. Apparent surface wear.

Each of these conditions was rated in terms of serviceability indices ranging from 1.0 to 5.0. In many cases, this information was important to the interpretation of the field and laboratory test data.

Laboratory Test Procedures

Cutting Core Specimens

Each pavement core was cut into five subdivisions with a concrete saw. These subdivisions included the surface course, leveling course, and top, middle and bottom thirds of the base course materials. Only the asphalt bound pavement materials were included in these subdivisions. Separation of the cores into the respective layers was done so that the specific gravities of the individual layers could be compared and differences in densities detected.

Bulk Specific Gravities

After a preliminary series of tests were conducted, it was concluded that the extent of water absorption of the field core sample sections was high enough to appreciably affect their observed bulk specific gravity values. Based on this finding, all of the core samples were tested for bulk specific gravity according to ASTM method of test, D 1188.

Maximum Specific Gravities

The theoretical maximum specific gravity of the asphalt-aggregate mixtures contained in the respective layers of the core specimens was determined using ASTM method of test, D 2041. The standard procedures were slightly modified so that existing laboratory equipment could be used. For details of these modifications, see Interim Report II (17).

Percent density values and percent air void contents of the core layers were calculated using the standard relationships for these quantities. Comparison of the percent density values of corresponding layers from core samples obtained from wheelpath and non-wheelpath locations was made. Significant differences in these values were attributed to the effects of traffic load densification.

Stereo-Photo Interpretation

In surface mixtures incorporating non-polishing aggregates, only the fine materials and the asphalt binder tend to wear or abrade under traffic. In mixtures made with polishing aggregates, both the surface aggregate particles and the background fines tend to abrade but at different rates, depending on the relative wear-resistance of the surface materials. The amount of wear due to traffic action, in either case, is greater on heavily traveled areas (wheelpaths) and the projections of the coarser surface particles above the matrix should be relatively higher than in less frequently traveled areas (non-wheelpath locations). Thus, comparision of the surface aggregate projections in the wheelpaths and outside the wheelpaths was considered to be a reasonably good way to estimate the amount of wear that had occurred.

Stereo-slide pairs were viewed under a telescopic lens with six power magnification on a flourescent light desk. By comparing the heights of projections with that of a calibrated wedge scale (placed on the pavement at the time the photographs were taken), the projection heights of the aggregate particles were recorded in terms of the scaled wedge height. These scale values were later converted to inches by means of a calibration curve.

Results and Discussion

Density Measurements

Information concerning the selected test sites is presented in Table X. Since initial or "as constructed" density values of the test site pavements were unknown, statistical methods were employed to test for differences in the observed percent density values for the respective layers at each transverse test point. Significant point differences were considered due to additional densification from traffic. Statistical information from the test site data was developed using the Statistical Analysis System (SAS) computer program (21). A value of 0.10 was used as the significance criterion for the acceptance or rejection of the hypothesis of no differences.

Plots of laboratory determined percent density values for the respective asphalt bound layers versus transverse location of test points were made for each test site. Fig. 4 shows such a plot for test site 70 on HMSA base. Generally, peak values of the surface, leveling and base course curves occurred at the wheelpath locations in both traffic lanes. This indicated that differential densification (the differences in wheelpath percent density value and the average of the two

TABLE X

TEST SITE INFORMATION

TEST SITE #	LOCATION	SECTION TYPE	BASE TYPE	AGE (MONTHS)	LOCATION OF MARKER (ORANGE PLATE)
10	Interstate 40, Westbound Lanes, Muskogee,County, Oklahoma, approx. 12.50 miles East of Mc-Intosh County line	Fill	Hot Mix Sand Asphalt	36	North right-of-way fence
20	Interstate 40, Westbound Lanes, Seminole County, Oklahoma, approx. 2.4 miles East of Pottawatomie County line	Fill	Black Base	82	North right-of-way fence
30	Interstate 40, Westbound Lanes, Sequoyah County, Oklahoma, approx. 13.0 miles East of Muskogee County line	Cut	Black Base	56	North right-of-way fence
40	Interstate 40, Eastbound Lanes, Sequoyah County, Oklahoma, approx. 12.6 miles East of Muskogee County line	Fill	Black Base	56	South right-of-way fence
50	Interstate 40, Westbound Lanes, Seminole County, Oklahoma, approx. 4.5 miles East of Pottawatomie County line	Slight Cut	Black Base	86	North right-of-way fence
60	Interstate 40, Westbound Lanes, Beckham County, Oklahoma, approx. 19.0 miles East of Texas-Oklahoma State line	Slight Fill	Hot Mix Sand Asphalt	169	North right-of-way fence
70	Interstate 40, Westbound Lanes, Beckham County, Oklahoma, approx. 20.50 miles East of Texas-Oklahoma State line	Slight Fill	Hot Mix Sand Asphalt	169	North right-of-way fence
80	Interstate 35, Southbound Lanes, Kay County, Oklahoma, approx. 25.0 miles North of Noble County line	Slight Fill	Stabilized Aggregate Base Course	165	West right-of-way fence
90	Interstate 35, Southbound Lanes, Kay County, Oklahoma, approx. 26.0 miles North of Noble County line	Slight Fill	Stabilized Aggregate Base Course	165	West right-of-way fence
100	Interstate 35, Southbound Lanes, Cleveland County, Oklahoma, approx. 4.5 miles North of McClain County line	Slight Fill	Stabilized Aggregate Base Course	158	West right-of-way fence
110	Interstate 40, Westbound Lanes, Cleveland County, Oklahoma, approx. 12.0 miles East of Oklahoma County line	Slight Fill	Stabilized Aggregate Base Course	158	West right-of-way fence
120	Interstate 40, Eastbound Lanes, Pottawatomie County, Oklahoma, approx. 12.0 miles East of Oklahoma County line	Slight Fill	HMSA	105	North right-of-way fence
130	Interstate 40, Eastbound Lanes, Washita County, Oklahoma, approx. 9.0 miles East of Beckham County line	Slight Fill	Soil-Cement Base	148	South right-of-way fence
140	Interstate 40, Westbound Lanes, Washita County, Oklahoma, approx. 12.25 miles East of Beckham County line	Fill	Soil-Cement Base	148	North right-of-way fence
170	Interstate 40, Eastbound Lanes, Beckham County, Oklahoma, approx. 26.4 miles East of Texas-Oklahoma State line	Fill	Soil-Cement Base	169	South right-of-way fence
180	Interstate 40, Eastbound Lanes, Beckham County, Oklahoma, approx. 20.25 miles East of Texas-Oklahoma State line	Fill	Soil-Cement Base	196	South right-of-way fence

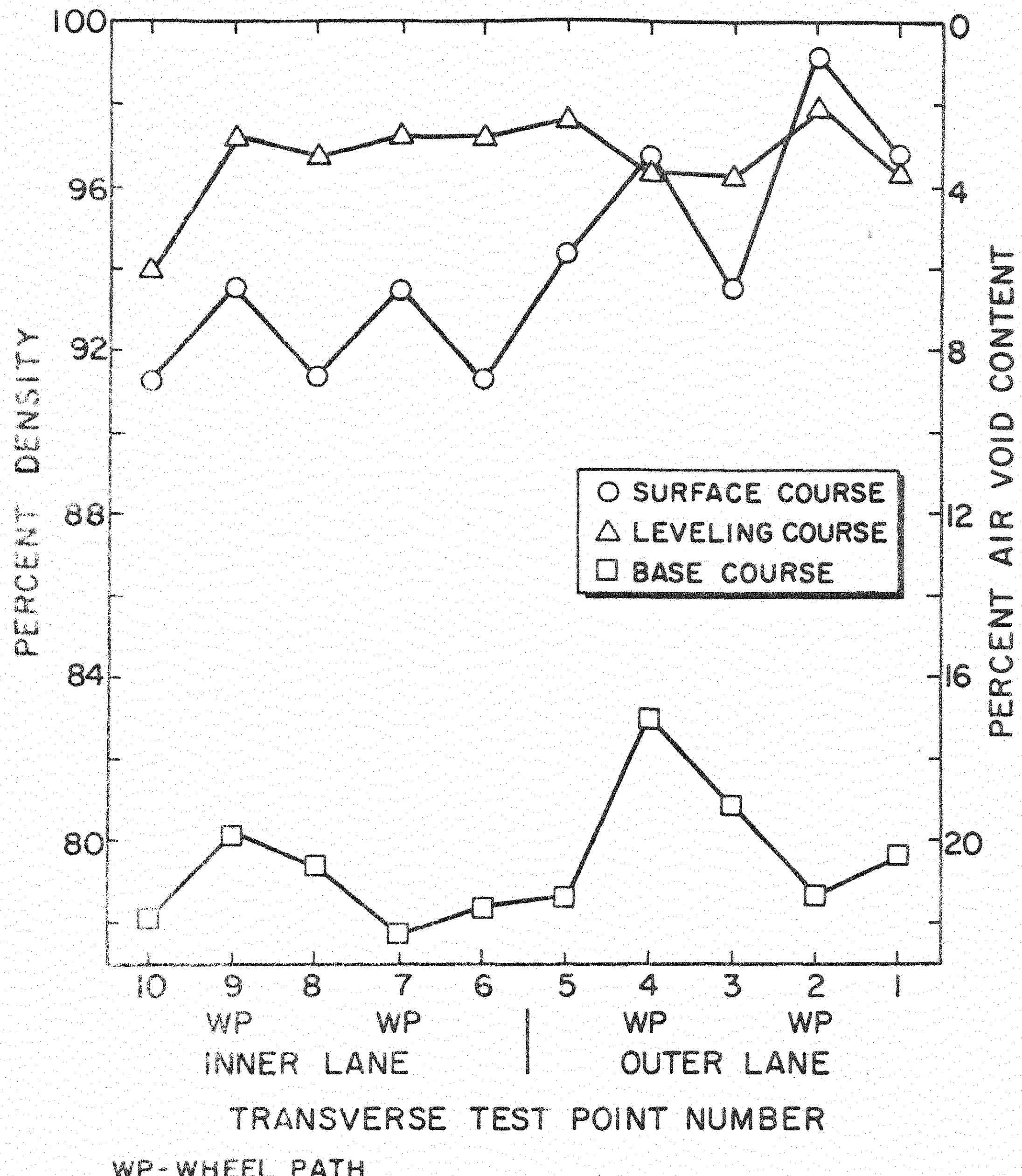

Figure 4. Percent Density Versus Transverse Test Point
Site #70, Base Type--HMSA

adjacent non-wheelpath density values) had taken place in all of these asphalt bound layers. The analysis of variance (AOV) of the percent density values also indicated strong evidence of point differences at these test sites.

For the most part, the percent density values of the surface and leveling courses at the sites were within the Oklahoma Department of Transportation specified limits for these materials and ranged from 99 to 82 percent. The percent density values of the HMSA bases showed a greater variation (ranging from 93 to 77 percent) and were considerably lower than those of the black bases.

Interestingly, the density of the leveling course was frequently greater than that of the surface layer. This condition was found at nine of the sixteen test sites. While this may be the result of inadequate compaction of the surface course material during construction, it can also be an indication that some decompaction is occurring in the surface layer from the kneading action of vehicle tires, particularly, if the stability of the base material is questionable. Low density and/or stability in surface course materials results in poor load-distribution characteristics. This causes a higher concentration of traffic stresses and encourages densification and lateral creep of the underlying materials.

Generally, the larger amounts of differential densification in the respective pavement layers occurred in the wheelpaths of the outer traffic lane, which received the greater number of wheel load applications. The type of base course influenced the amount of densification, e.g., minimum amounts of densification in the surface and leveling courses occurred on the SCB materials with larger amounts evidenced in the same layers on HMSA and SABC materials. Additionally, a considerable amount of differential densification took place in the HMSA bases.

Nuclear Density Measurements

Laboratory determined densities of the surface layer from the cores taken at the transverse test points on the pavement were compared with the corresponding values obtained with the nuclear density gage at these same points. There was little or no correlation between the two sets of data.

Although some error in the laboratory density values was likely, the poor correlation was largely attributed to the nuclear density measurement technique. The accuracy of the backscatter method was adversely affected by surface roughness. Even with the most careful precautions to insure representative density counts with the gage, the determined densities were frequently so low that they were considered unreliable.

Surface Wear Measurements

The surfaces at many of the test sites showed substantial amounts of dislodgement of the coarse aggregate particles from the pavement matrix and often this condition had progressed to the "raveling" stage. This weakened the procedure used to determine the depth of surface wear since it involved estimating wear by comparing the projection height of surface aggregates and did not account for the depth of material lost by dislodgement.

Table XI shows a summary of the results of the stereo-photo

TABLE XI

DIFFERENTIAL WEAR

BASE COURSE MATL.	SITE #	AGE MO.	AVERAGE DIFFERENTIAL WEAR (INS.)[1] AT WHEELPATH LOCATIONS	
			INNER LANE	OUTER LANE
HMSA	10	36	0.012	0.020
	60	169	0.016	0.023
	70	169	0.018	0.027
	120	105	0.016	0.029
BB	20	82	0.008	0.071
	30	56	**	**
	40	56	**	0.016
	50	86	0.015	0.016
SABC	80	165	0.016	0.031
	90	165	0.023	0.078
	100	158	**	**
	110	158	**	**
SCB	130	148	0.010	0.017
	140	148	0.005	0.014
	170	169	0.028	0.039
	180	169	0.031	0.031

** MISSING VALUES

1. CONVERSION: 1.0 in. = 2.54 cm

interpretation for each of the test sites. The tabulated values are averages of the amounts of differential wear determined at the wheel-paths locations for a given lane. As would be expected, the larger values occurred in the outer lane. Missing data reflects poor quality photographs resulting from a malfunction of the camera. While the magnitudes of wear were smaller than anticipated, the results indicated that wear was definitely a contributing factor to pavement rutting.

Profile Measurements

As discussed earlier, a transverse profile trace or graph was made for each pavement lane at a given test site and the rut depths and/or upheaval of the surface scaled from these traces. The two graphs for each site were put together and reduced photographically for presentation. Vertical and horizontal scales were indicated and the graphs oriented as they would appear when viewed in the direction of traffic on the pavement. The scaled rut depths (positive values) and the measurements of heave (negative values) were shown immediately below their respective locations on the tracing.

The transverse profile tracing for test site 10 is shown in Fig. 5 . The pavement was approximately 3 years old and consisted of a 2.0 in. (5.08 cm) Type C surface course, a 2.5 in. (6.35 cm) Type B leveling course and 9.0 in. (22.86 cm) of HMSA base on a slight fill section. Based on the previously stated assumptions concerning the method of measurement, heave was indicated at test points 1, 3, and 8, and the maximum rut depth of 0.99 in. (2.52 cm) occurred at test points 2 and 4. The density curves for this site showed relatively high and uniform densities of about 98 percent in the surface and leveling courses and an average of about 90 percent in the base course.

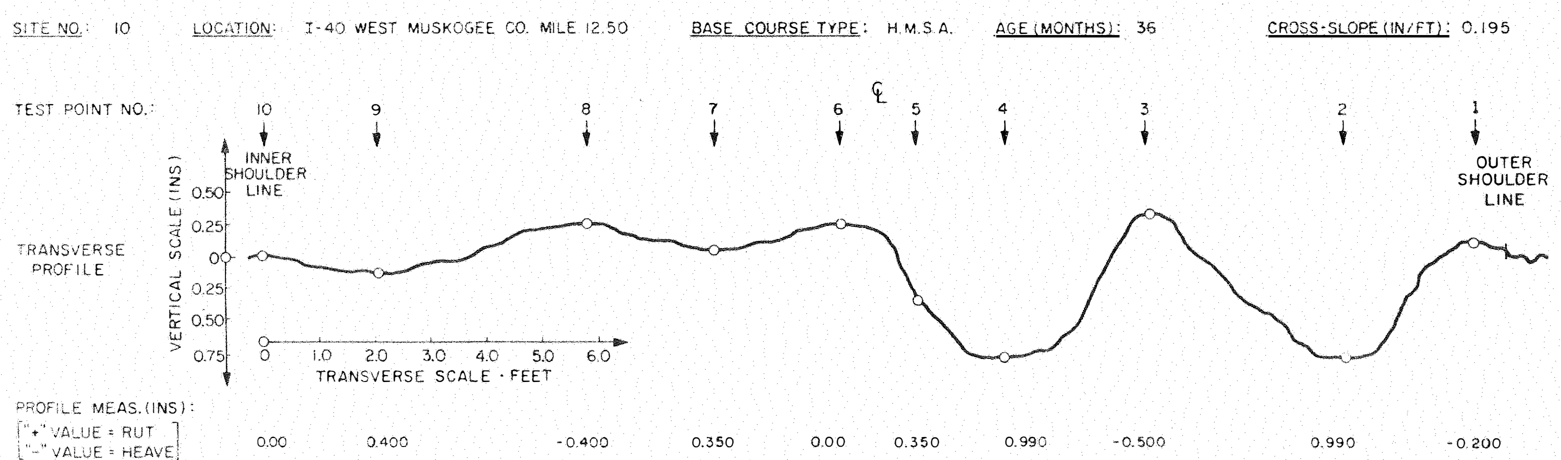

Figure 5. Transverse Profile Tracing, Test Site 10

At test point 2, the observed increase in density, i.e., the differential densification, in all of the pavement layers accounted for approximately 0.28 in. (0.71 cm) of the total rut depth. This is based on the fact that the increase in percent density of a laterally confined column of paving mixture is directly proportional to the percent change in height of the column. The average amount of surface wear in the outer lane was 0.02 in. (0.05 cm). The remaining 0.69 in. (1.75 cm) of the total rut depth at this point was attributed to lateral creep of material in one or more of the layers. The adjacent surface heaves show that a substantial amount of displacement has occurred. In this case, lateral creep, densification, and surface wear accounted for 70, 28, and 2 percent of the observed rutting, respectively.

Similar traces and analyses were made for each test site. Table XII shows the contributions of the various modes of rutting at the respective test sites. The results showed that the bituminous paving mixtures were responsible for a significant amount of the rutting that had occurred. That is, the lack of proper densification and high stability in the respective pavement layers were conducive to the development of excessive rut depths.

State Highway 51 Measurements

Measurements at two test sites on a newly constructed segment of State Highway 51 were made over a two year period. These sites were designated SH 51-2 and SH 51-3. Transverse profile tracings, core samples, nuclear densities and stereo-photographs were obtained at these sites prior to opening of the new highway section to traffic. The information developed from this study added to and substantiated the

TABLE XII

MODAL CONTRIBUTIONS TO RUTTING

Site No.	Base Course	Age (mo.)	Max. Rut (in.)	Approximate Percentage Contribution to Rutting			
				Densification	Surface Wear	Lateral Creep	Base or Subgrade Deformation
10	HMSA	36	0.990	28%	2%	70%	*
60	HMSA	169	0.429	40%	5%	55%	*
70	HMSA	169	0.574	40%	5%	55%	*
120	HMSA	105	0.380	42%	8%	50%	*
20	BB	82	0.571	58%	12%	30%	*
30	BB	56	0.607	25%	--	*	73%
40	BB	56	0.643	22%	2%	*	76%
50	BB	86	0.786	8%	2%	*	90%
80	SABC	165	0.571	9%	5%	*	86%
90	SABC	165	0.625	9%	13%	*	78%
100	SABC	156	0.607	18%	--	*	77%
110	SABC	156	0.482	20%	--	*	74%
130	SCB	148	0.219	43%	8%	49%	
140	SCB	148	0.289	30%	5%	65%	
170	SCB	169	0.500	14%	8%	78%	*
180	SCB	169	0.478	25%	6%	69%	*

* Not a major factor - some contribution indicated

previously discussed results of the interstate study.

Figs. 6 and 7 show plots of maximum rut depth versus time for the traffic lanes at the SH 51-3 test site. The curves show that the rate of rutting was greatest during the first 4 to 6 months, with maximum rut depths occurring between the 8th and 10th month and a gradual reduction in rut depth thereafter. Deeper ruts occurred in the inner wheelpaths and this was also observed at a majority of the interstate study sites. The deeper ruts in the westbound lane are indicative of higher traffic volumes and heavier wheel loads in this direction.

Following construction, both the subgrade and the asphalt bound paving materials are most susceptible to traffic densification and lateral displacement under traffic loads. If the initial density and stability of the pavement layers are low enough, ruts develop rapidly during the first few months of exposure to traffic loading. As time progresses, post-construction densification increases the resistance of the materials to vertical or lateral displacement and the rate of rut development declines.

Subsequently, a gradual reduction in rut depth takes place. The major cause of this reduction is attributed to the tendency of motorists to shift the path of their vehicles to avoid driving in deep ruts. This lateral translation of vehicle wheels creates a widening of the wheelpath depressions. The resulting subjugation or flattening of the transverse curvature in the pavement surface reduces the measured depth of rut.

The AOV of the density values determined for the respective layers of the core samples at these sites indicated strong layer, point and time differences. Fig. 8 illustrates the densification trends in the respective layers at the SH 51-3 site. Each of the layers experienced a high

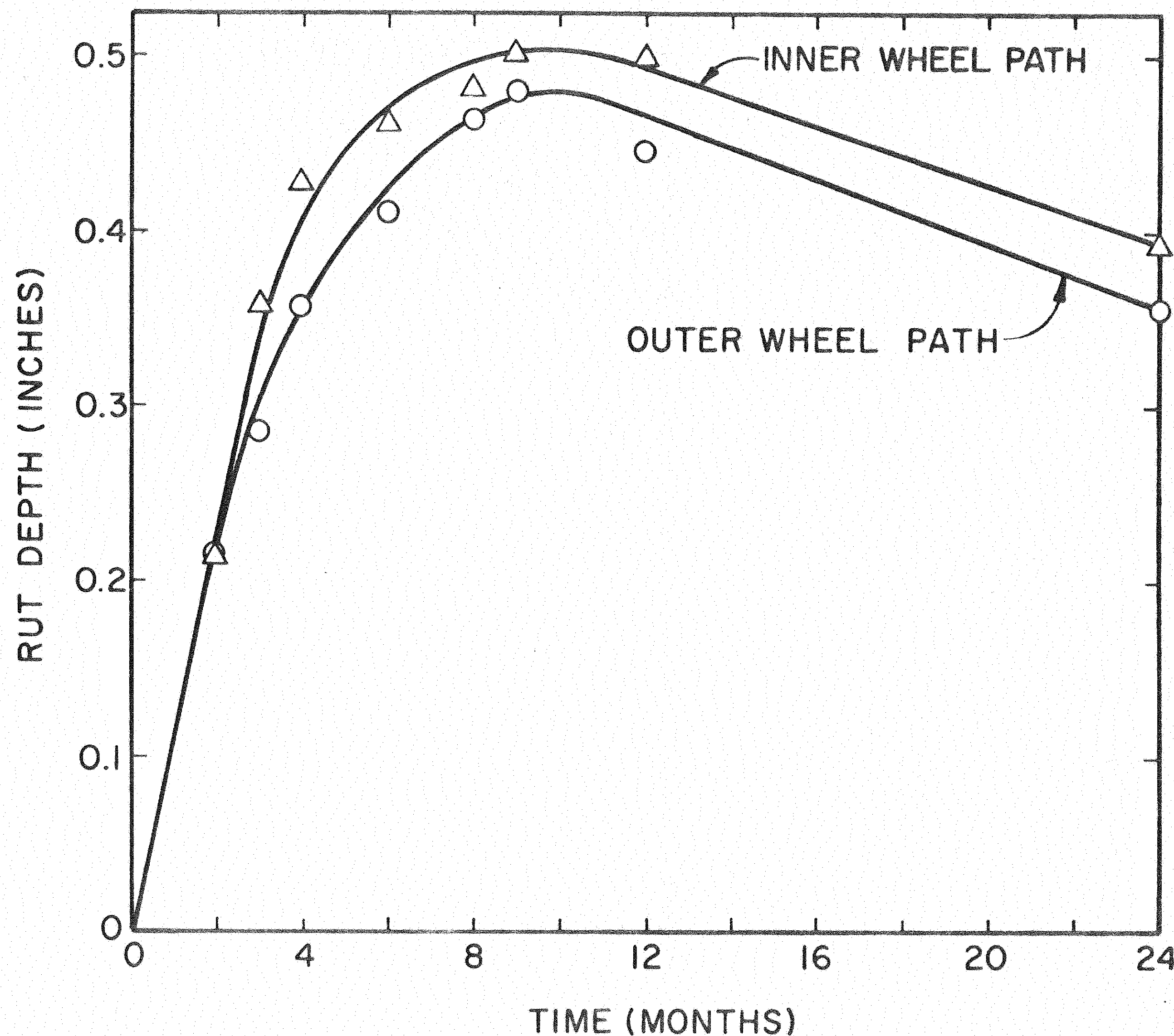

Figure 6 . Maximum Rut Depth Vs. Time, SH 51-3, Eastbound Lane

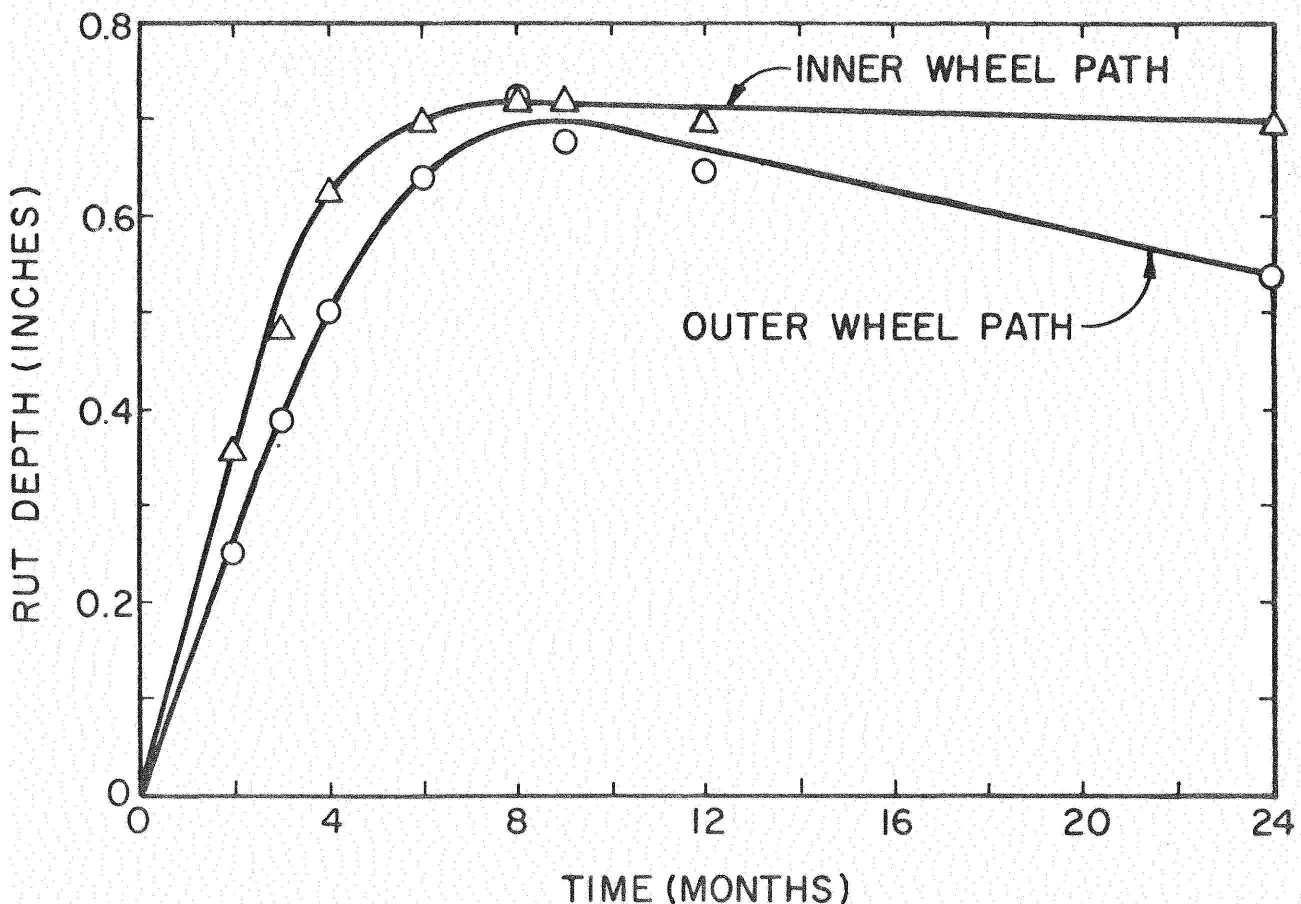

Figure 7. Maximum Rut Depth Vs. Time, SH 51-3, Westbound Lane

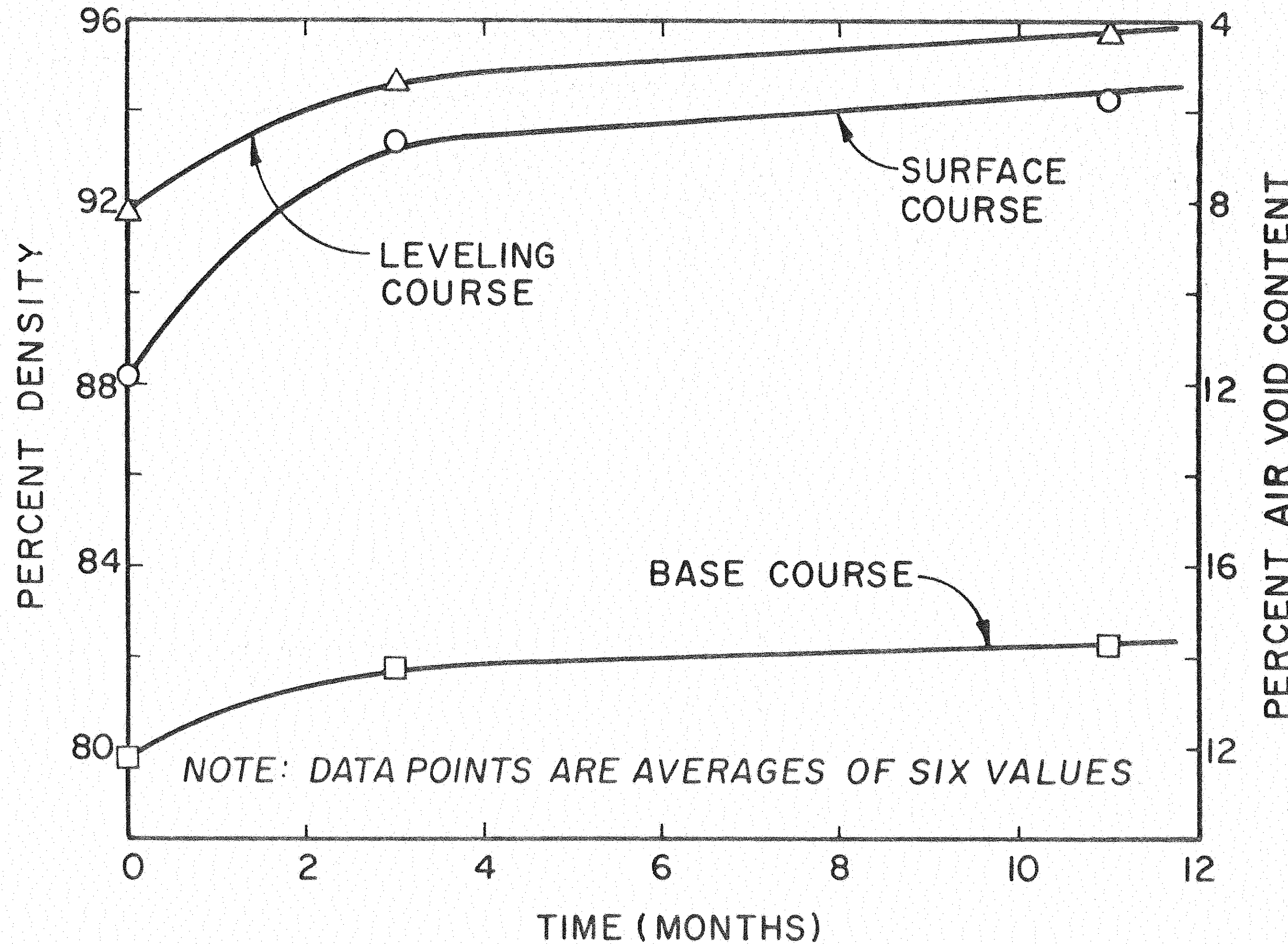

Figure 8. Percent Density Vs. Time, SH 51-3 Test Site

rate of densification during the early stages of traffic exposure and then a subsequent reduction. As in the interstate sites on HMSA base, the leveling course had the highest density and the surface course material, due to its initial low density and the higher vertical stresses at the pavement surface, exhibited the greatest increase in density.

The pavement surfaces at the two test sites experienced substantial amounts of vertical translation during the study period. Because of this, no datum could be established for the measurement of surface heave adjacent to the wheelpaths. A considerable amount of lateral creep was suspected, however, due to the appearance of the pavement surfaces and the extremely low density of the hot-sand base.

Based on the 8th month rut depths and interpolated density values (at this point in time) from the density-time plots, densification was responsible for 56.3 percent of the rut depth at the SH 51-3 site and 45.8 percent of the rut depth at the SH 51-2 site. Stereo-photo analysis indicated that surface wear was minimal at both locations so the balance of the maximum rut depths was considered due to lateral creep or instability of the respective materials layers. Most of this probably occurred in the low density base. The direct relationship between the density and stability of a mix (22) lends credence to this analysis.

Conclusions

Based on the test procedures used and the results from the pavement sections studied in this part of the investigations, the following conclusions are made:

1. Post-construction densification from traffic loads occurred in all asphalt-bound material layers of the pavement sections studied. In the surface course layer, increases of up to 10 percent in density were observed at wheelpath locations.

2. Densification contributed significantly to the total rut depths measured at the test sites. From 8 to 58 percent of the rut depth on pavements constructed on asphalt bases was attributed to densification.

3. From observations made at newly constructed pavement sections, ruts developed rapidly during the first four to six months with maximum rut depths occurring approximately nine months after the pavement was opened to traffic. Subsequently, a gradual reduction in rut depth was noticed.

4. Surface wear or attrition in the wheelpaths on heavily traveled pavements was an important contributing factor to rutting. Raveling or dislodgement of surface aggregate particles hampered the determination of amount of wear from stereo-photographs.

5. Evidence of lateral creep or instability of the bituminous material layers was found at a majority of the test sites. More prominent surface heaves were associated with pavement sections where the asphalt-bound layers had very low densities. Up to 78 percent of the rut depth at specific sites was considered due to lateral creep.

6. The transverse profile gage developed for this study provided tracings of a pavement surface from which accurate measurements of surface deformations could be made. This type of instrument will be a valuable research tool in future studies of flexible and rigid pavements.

CHAPTER IV

TRANSVERSE CRACKING IN FLEXIBLE PAVEMENTS

Transverse cracking in the surface course of flexible pavements is one of the serious highway performance problems in Oklahoma. Many factors, such as asphalt and aggregate properties, mix design, construction procedures, environmental conditions, and traffic loading influence the ability of a pavement to resist cracking (23, 24, 25). This form of cracking impairs the riding quality and shortens the life of the pavement.

Previous field and laboratory studies have indicated that transverse cracking is most likely to occur due to thermal tensile stresses that are developed by pavement contraction at low temperature (25). These cracks appear in the surface when the accumulated tensile stresses exceed the fracture strength of the asphalt concrete pavement. These studies also showed that the rheological properties of the asphalt binders largely affect the ability of asphalt concrete surfaces to resist this type of cracking (25, 26).

The major objectives of this portion of the research project was to determine the nature and extent of these cracks in Oklahoma flexible pavements and to investigate the possible causes of this form of distress. The research dealt primarily with the bituminous components of the pavement and their influence or contributions to transverse cracking.

Nine pavement test sections, 500 ft (152.4 m) in length, with various degrees of cracking were selected for study. Crack mapping and counting techniques were used to determine the severity of cracking at each test site. Six inch (15.24 cm) diameter cores of the asphalt-bound paving materials were obtained at locations along newly developed transverse cracks to determine the extent or depth of crack penetration in the pavement. Four inch (10.16 cm) diameter cores were taken at random locations in the vicinity of the cracks to provide samples of the field asphalt cements and mixtures for laboratory testing.

A tensile splitting apparatus was developed to evaluate the tensile properties of asphalt paving mixtures at various low temperatures. This test apparatus was employed to investigate the tensile properties of specimens obtained from field cores and laboratory prepared specimens containing various asphalt contents and asphalts having different viscosity characteristics.

The asphalt binders were recovered from the respective core specimens and tested to evaluate their rheological properties. The "stiffness modulus" concept (27) was used to characterize the behavior of the asphalt cement samples and mixtures at low temperatures. The Statistical Analysis System (SAS) computer program (21) was used to analyze the test results and detect correlations between the results and evidenced field behavior.

Tensile Splitting Test Apparatus

A relatively simple and practical tensile splitting apparatus was developed to determine the tensile properties of 4.0 in. (10.16 cm) diameter field and laboratory specimens at various low temperatures. This apparatus was patterned after similar equipment used by Hudson (28) and is shown in Fig. 9. Details of the test apparatus and its operation are presented in Interim Report V (29).

The tensile splitting test used in this study involved applying compressive loads at a rate of 0.06 in./min (0.15 cm/min) across a diameter of cylindrical asphalt concrete specimens. A load cell placed on the upper loading plate of the apparatus continuously measured the applied compressive load. Horizontal deformation of the specimen was determined by series-connected linear variable differential transducers on opposite sides of the specimen. Output signals from the load cell and the transducers were conveyed to an X-Y recorder, which plotted a continuous load-deformation trace. From this graph, the maximum load at failure (P_{max}) and the corresponding total horizontal deformation (X_{TF}) were determined. Using these values, the tensile stress and tensile strain at failure of the specimens were calculated. The ultimate failure stiffness (S_{TF}), i.e., the ratio of the failure stress to failure strain, was also calculated. Fig. 10 shows a typical load-deformation trace and the resulting calculated values for a pavement core specimen.

Field Test Procedures

Site Selection

A total of nine pavement test sections having a wide variation in the degree or severity of transverse cracking were studied. Four of the sites selected for study were on State Highway 177 and the other five sites were on sections of Interstate 35 and Interstate 40. Two of the interstate highway sites had little or no transverse cracking and were chosen for purposes of comparison.

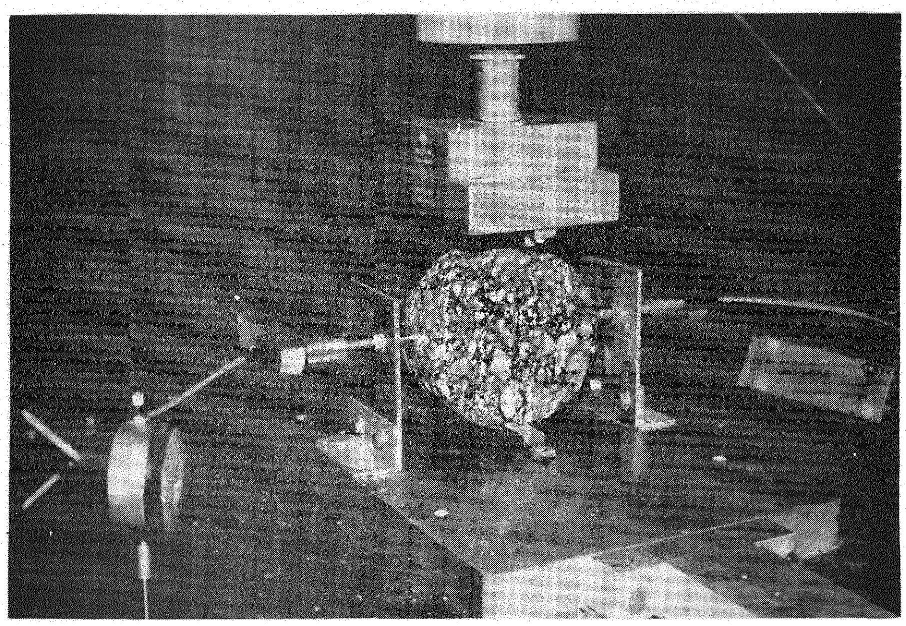

Figure 9. The Modified Tensile Splitting Test Equipment

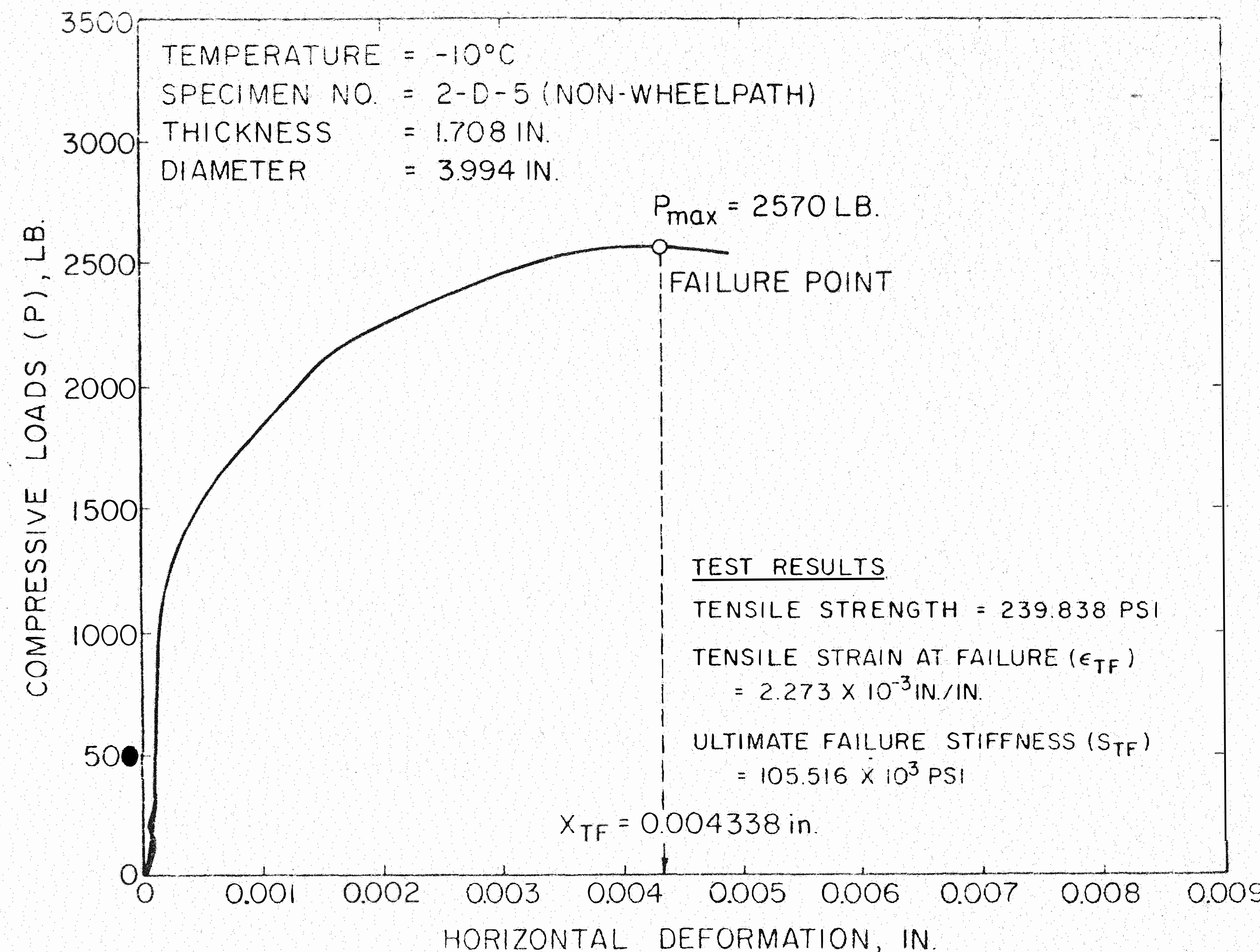

Figure 10. A Typical Load-Deformation Graph from the Indirect Tensile Splitting Test

At each test site, a 500 ft (152 m) length of pavement satisfying established safe sight distance requirements was selected for detailed crack surveying and coring operations. The beginning and ending points of these 500 ft lengths were marked with yellow paint stripes along the shoulders.

Crack Survey

The crack survey included mapping and counting all of the transverse cracks encountered in a 500 ft pavement length chosen for study. Crack patterns were sketched on an appropriate field data sheet and the cracks were classified according to type (multiple, full, half and part). The number of these respective types of cracks at each site was used to calculate the cracking index (C.I.) of the section as a measure of the severity of cracking that had occurred (30). These data are shown in Table XIII.

At each site, newly developed cracks, i.e., extremely narrow transverse cracks that did not extend the full width of the pavement, were selected and marked during the crack survey. Six inch (15.24 cm) diameter pavement cores spanning these narrow cracks were obtained to determine the depth of the cracks and the possible mechanism of transverse cracking.

Core Drill Operations

In addition to the large diameter cores taken to study the depth of beginning cracks, a number of 4.0 in. (10.16 cm) diameter full depth paving cores were obtained at each site for laboratory testing. Because of traffic densification of the asphalt paving materials, these no page 59 in document

cores were obtained at both wheelpath and non-wheelpath locations on the pavement. The tensile properties of these field samples were to be evaluated at three low temperatures. Consequently, eighteen field cores were required at each site. Randomization principles were used to select three full width cracks at each of the individual test sites. At an offset distance of approximately 8.0 in. (20.32 cm) from the edges of these cracks, six core specimens were obtained as shown in Fig. 11. Each specimen was wrapped in plastic, appropriately identified and carefully stored for transporting to the laboratory.

Laboratory Test Procedures

Field Core Specimens

The large diameter cores that cut across the very narrow transverse cracks were carefully examined and photographed in the laboratory. Examination revealed that in a majority of these core samples the "beginning" cracks did not extend through the pavement matrix. Apparently, these cracks had originated at the surface and had propagated to only a limited depth in the underlying layers. These findings substantiated those of previous investigators and directed the subsequent research work toward investigating the behavior of the surface asphalt materials and mixtures at low temperatures.

The pavement specimens used in the tensile splitting tests were obtained by cutting the surface layer, approximately 2.0 in. (5.08 cm) thick, from the 4.0 in. (10.16 cm) diameter cores with a concrete saw. The dimensions of these specimens were measured and their bulk specific gravities determined prior to testing. After performing the tensile splitting test, the theoretical maximum specific gravity of the surface

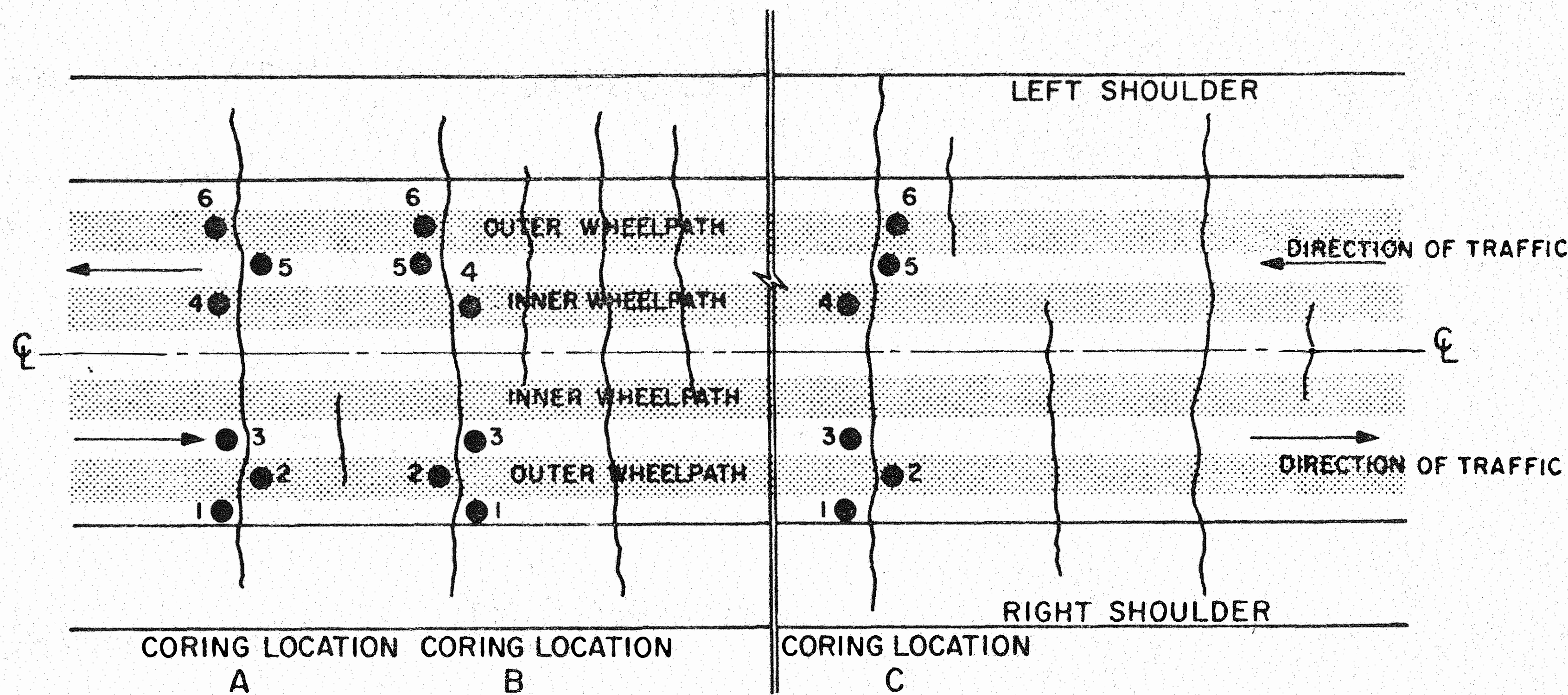

Figure 11. A General Layout of Coring Plan for a 2-Lane Test Site

mixtures in the field specimens were obtained using ASTM method of test, D 2041. These specific gravity values were used to calculate the percent density and air void content of the wheelpath and non-wheelpath materials.

Laboratory Specimens

Eighty-one laboratory compacted Hveem-Gyratory specimens of a Type C surface course mixture (4) were prepared and tested. It was desired to study the effect of three factors, temperature, asphalt content and asphalt penetration grade, on the tensile properties of these asphalt concrete mixtures. Three levels of each factor were employed to permit useful comparisons of the results. These factors and levels were as follows:

1. Temperature: +20°, 0°, and -10°C
2. Asphalt content: 4.5, 5.0, and 5.5 percent by total weight.
3. Asphalt grade: 91, 124, and 160 penetration.

Prior to performing the tensile splitting test on these specimens, their bulk specific gravities were determined and, following the test, theoretical maximum specific gravities of the specimen mixtures were determined.

Tensile Splitting Test

The dimensions, i.e., the thickness and diameter, of each test specimen were carefully measured to the nearest 0.001 in. (0.025 mm). In order to easily position a test specimen on the loading strips of the test apparatus, chalk was used to mark a diametral loading line on one face of the specimen. The specimens were then cooled in a freezer to the desired test temperature. Field core specimens were tested at temperatures of 0°, -5° and -10°C and laboratory compacted specimens at the previously stated temperatures.

Preliminary studies using dummy specimens with implanted thermistors and a remote sensing temperature monitoring device indicated that the average rate of temperature increase in a specimen removed from the freezer was 1°C/min over the range of temperatures used. It required approximately four minutes to remove a test specimen from the freezer, position it in the test apparatus and make the necessary equipment checks before starting the load application. Thus, the temperature of the specimen was adjusted to four degrees less than the actual test temperature and the load applied exactly four minutes after removing the specimen from the freezer.

A universal hydraulic testing machine with a 60,000 lb (27,216 kg) capacity was used to apply the compressive load at a head speed of 0.06 in./min (0.15 cm/min). As previously discussed, the applied compressive load and the corresponding horizontal deformation of a specimen was recorded by an X-Y recorder in the form of a continuous load-deformation trace.

Asphalt Extraction and Recovery

The asphalt binder in the surface course mixtures from the field cores was extracted in accordance with ASTM method of test, D 2172. Enough paving material to provide approximately 200 gm of recovered asphalt cement was used in these extractions. Surfacing specimens were randomly chosen from the field cores so that representative samples of asphalt cement would be obtained from each test site.

The asphalt was recovered from the extraction solution using a slightly modified Abson method, ASTM method of test, D 1856. The modifications of this standard method are explained in Interim Report V (29) and were developed to minimize the amount of hardening of the asphalt cement that occurred during the course of the recovery procedure.

Properties of Recovered Asphalt

Three samples of asphalt cement recovered from the surface mixtures at each test site were tested for certain physical and rheological properties. These tests included the penetration test (ASTM D 5), kinematic viscosity test (ASTM D 2170), absolute viscosity test (ASTM D 2171) and the ring-and-ball softening point test (ASTM D 36). These test values were used to calculate the stiffness moduli of the recovered asphalt cements according to McLeod's method (31).

Results and Discussion

Field Core Samples

Tensile Strength: The analysis of variance of all test results indicated strong evidence of location (wheelpath and non-wheelpath) differences in the tensile strength values. The tensile strengths of the wheelpath specimens were considerably greater at all test temperatures and this was attributed to the relatively higher pavement densities in the wheelpath locations. Also, the average tensile strengths were noticeably higher at -10°C than those at -5° and 0°C, respectively. This indicated the general behavior of the asphalt concrete mixtures at low temperatures, i.e., as temperature decreased, the mixture became increasingly rigid, lost some of its plasticity and behaved in an elastic manner.

A correlation study was made to investigate the general trend of the relationship between the tensile properties of the specimen and the cracking indices of the test sites. To minimize the effect of variation in material properties, this study was made separately on the test results of the wheelpath and non-wheelpath specimens. A Hewlett-Packard Calculator Plotter was used to plot the regression lines and the coefficients of correlation and determination (r and R^2) were computed for the first and second degree polynomials by the SAS computer program.

Fig. 12 shows the relationship between tensile strength of wheelpath specimens and the cracking index. Generally, test sites with a high degree of cracking had surface mixtures that exhibited lower tensile strength values. The results indicated that pavement surface mixtures with high tensile strengths will be more resistant to fracture at low temperatures.

Tensile Strain at Failure: The average values of tensile strains at failure for wheelpath specimens at all low temperatures were considerably higher than those of the non-wheelpath specimens. This, again, showed the effect of traffic densification and, apparently, the desirability of achieving a high density in the pavement surface of the time of construction. Generally, the tensile strain at failure significantly decreased as the test temperature decreased. It is interesting that a strong relation was observed between tensile strength and strain at failure. For example, specimens from Site 9 (C.I. = 0.5) exhibited the highest average tensile strength and strain values at failure while those specimens from Site 4 (C.I. = 24.5) had the lowest average tensile strength and strain values at failure.

The results of the correlation analysis indicated a strong

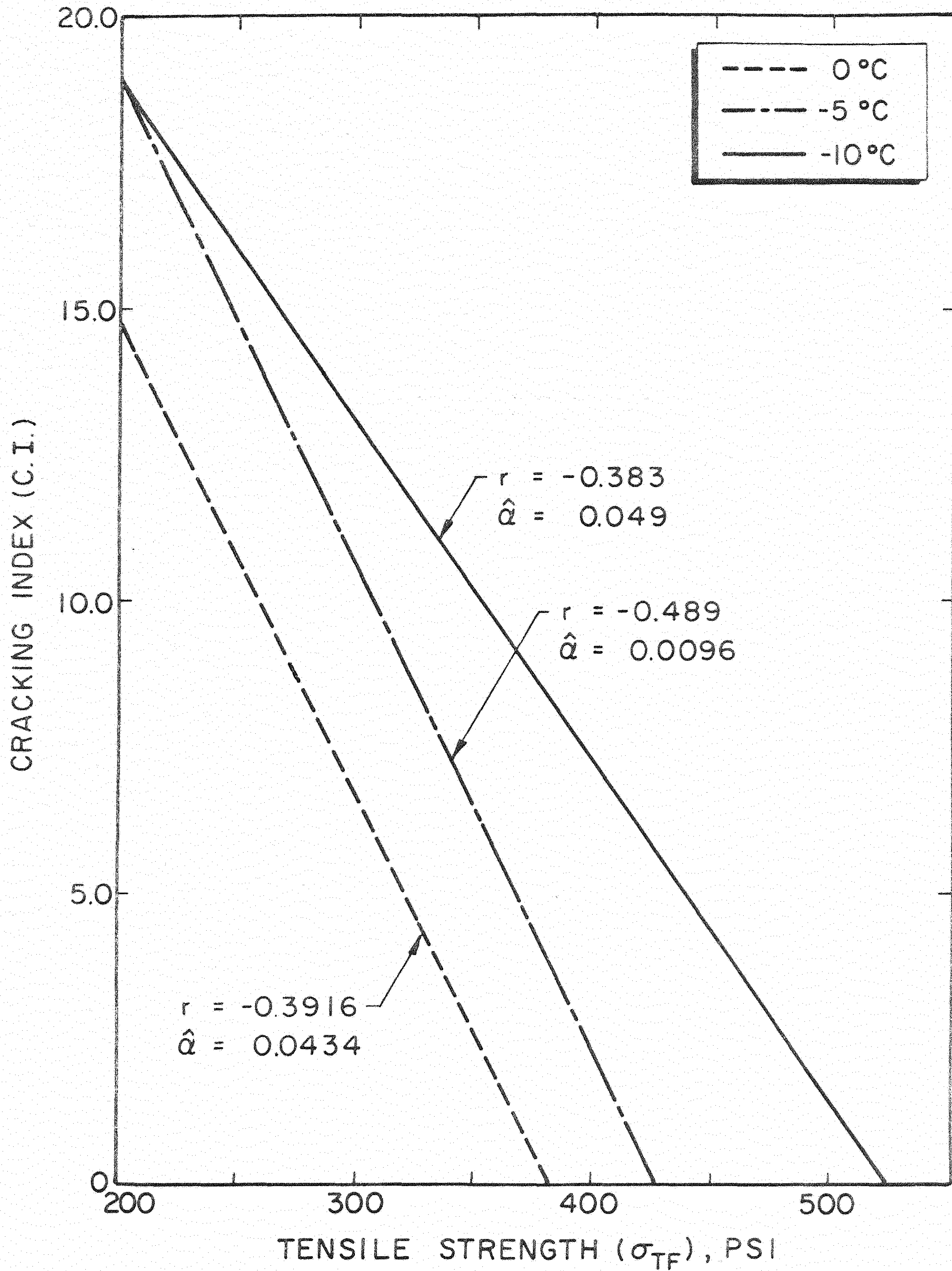

Figure 12. Relationship Between Tensile Strength of Wheelpath Specimens and Cracking Index

relationship between the tensile strains at failure and the observed degree of cracking. This relationship for wheelpath specimens is shown in Fig. 13. The occurrence of transverse cracking increased as the tensile failure strain decreased. This suggests that the resistance to cracking at any low temperature is a function of the strain capability of the asphalt concrete mixture. It also appears that a permissable or "standard" failure strain for mixtures used in a given geographical region could be established. Such a value could be used in mix design procedures to help alleviate transverse cracking.

Ultimate Failure Stiffness: In general, higher average failure stiffness values were observed at the lower temperatures. The correlation analysis indicated that the cracking index at a site was proportional to the ultimate failure stiffness of the mixes at all test temperatures. The test sites with a high degree of cracking or high C.I. usually had surface mixtures that exhibited the higher failure stiffness values.

Stiffness Moduli of Recovered Asphalt Cements and Field Mixtures: A study of climatological data (32) over a 73-year period showed that the lowest minimum air temperature recorded in Oklahoma was -17°F (-27.22°C). Based on temperature data reported in another research study (33), the temperature at a pavement depth of 2.0 in. (5.08 cm) could be expected to be about 7° to 8°F higher than the air temperature. Consequently, the minimum expected pavement temperature at a depth of 2 in. in Oklahoma was considered to be -10°F (-23.33°C). Stiffness moduli values of the recovered asphalt cements and the field mixtures were calculated at this temperature according to McLeod's method.

The low-temperature stiffness moduli of the recovered asphalt cements were correlated with the observed degree of cracking (Fig. 14).

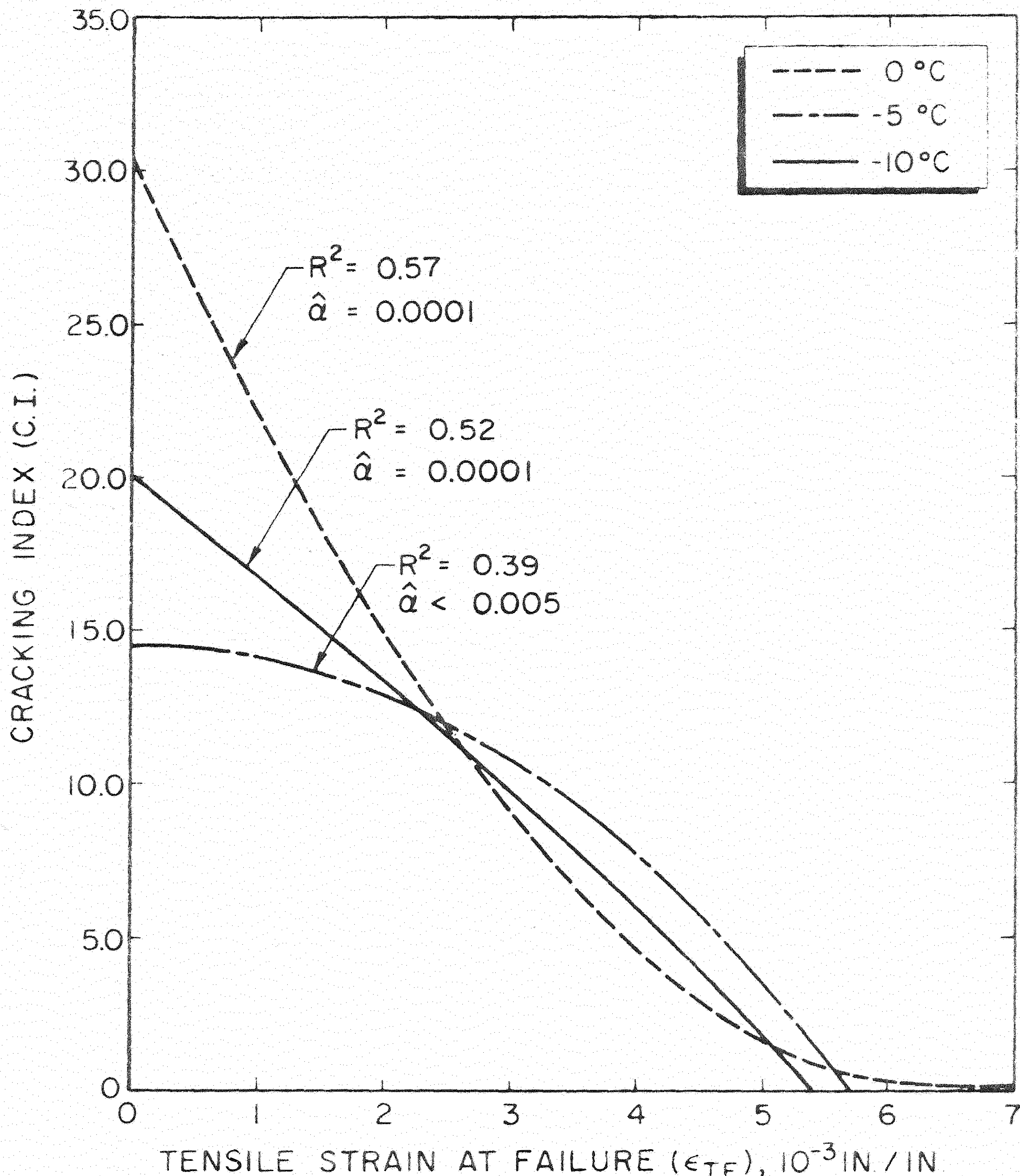

Figure 13. Relationship Between Tensile Failure Strain of Wheelpath Specimens and Cracking Index

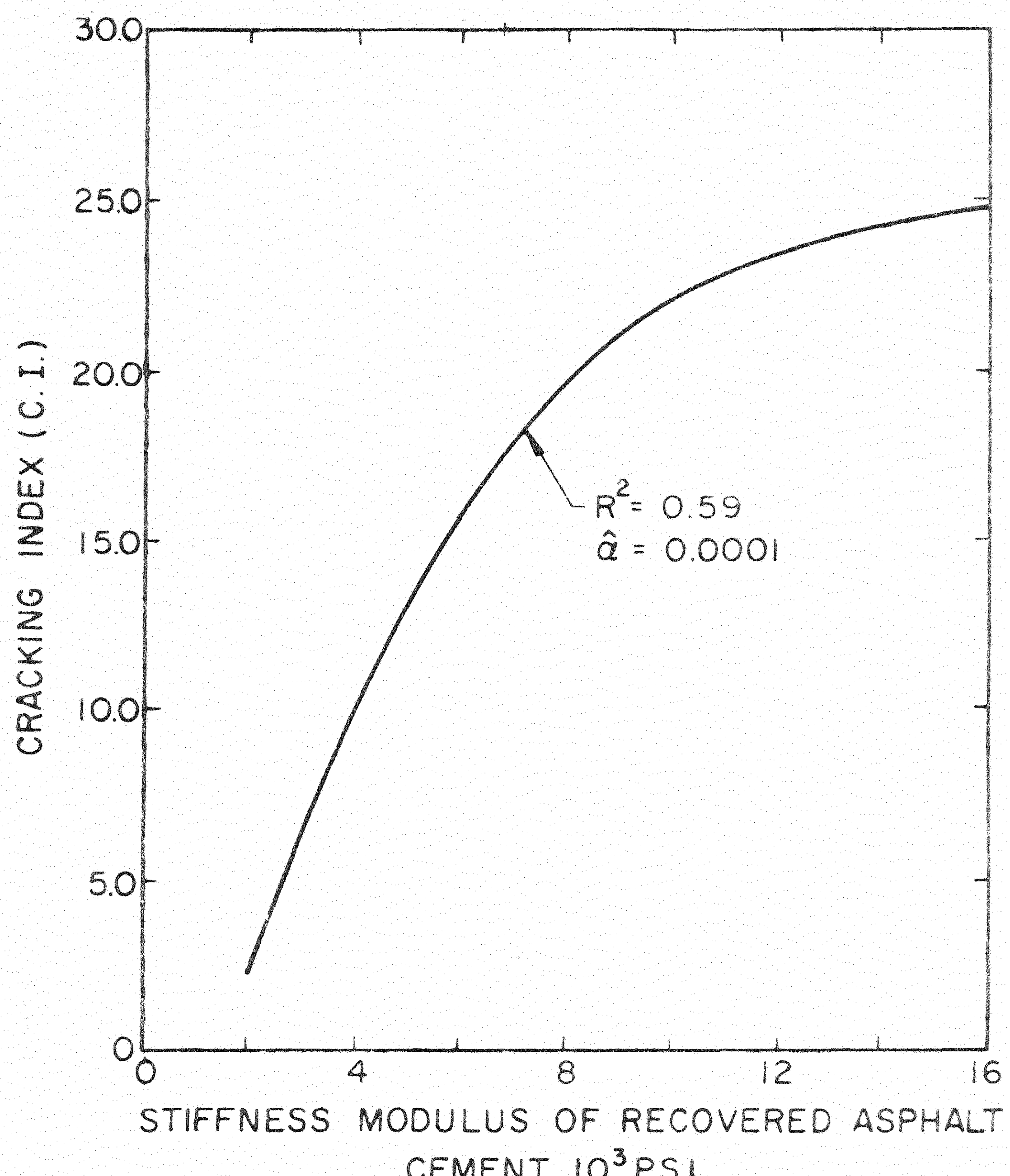

Figure 14. Relationship Between Stiffness Moduli of Recovered Asphalt Cements and Cracking Index

Pavement sections with a high C.I. were, generally, those having the stiffer asphalt cements in the surface layer. Fig. 15 shows the correlation between the calculated stiffness moduli of the mixtures from wheelpath and non-wheelpath specimens and the cracking index. The coefficients of determination for this relationship were relatively smaller and this was attributed to the variation in mix properties of the surfacing at the respective test sites.

Laboratory Mixtures

The results of the tensile splitting tests on the laboratory prepared specimens and the statistical analysis of these data were not particularly conclusive. This was attributed in part to the difficulties experienced in testing the laboratory specimens in the tensile splitting apparatus. Some modification in the arrangement for measuring horizontal deformation of the relatively softer and more deformable laboratory specimens appeared necessary.

Mixtures prepared with the 91-penetration asphalt cement showed the highest tensile strength values at all test temperatures and the strengths of the 124-penetration mixes were higher than those of the 160-penetration mixes at all temperatures. In all of the mixtures, the tensile strength or stresses developed at failure increased markedly as the temperature decreased. Asphalt content of the mixtures influenced the developed tensile strength but this effect was not consistent. Generally, the average tensile strength decreased as asphalt content increased.

The average tensile strains at failure of all the mixtures at 20°C were much higher than those at 0° and -10°C. This reflects the elastic response of the mixtures at low temperatures which would reduce the ability of a pavement to absorb the contraction strains developed at

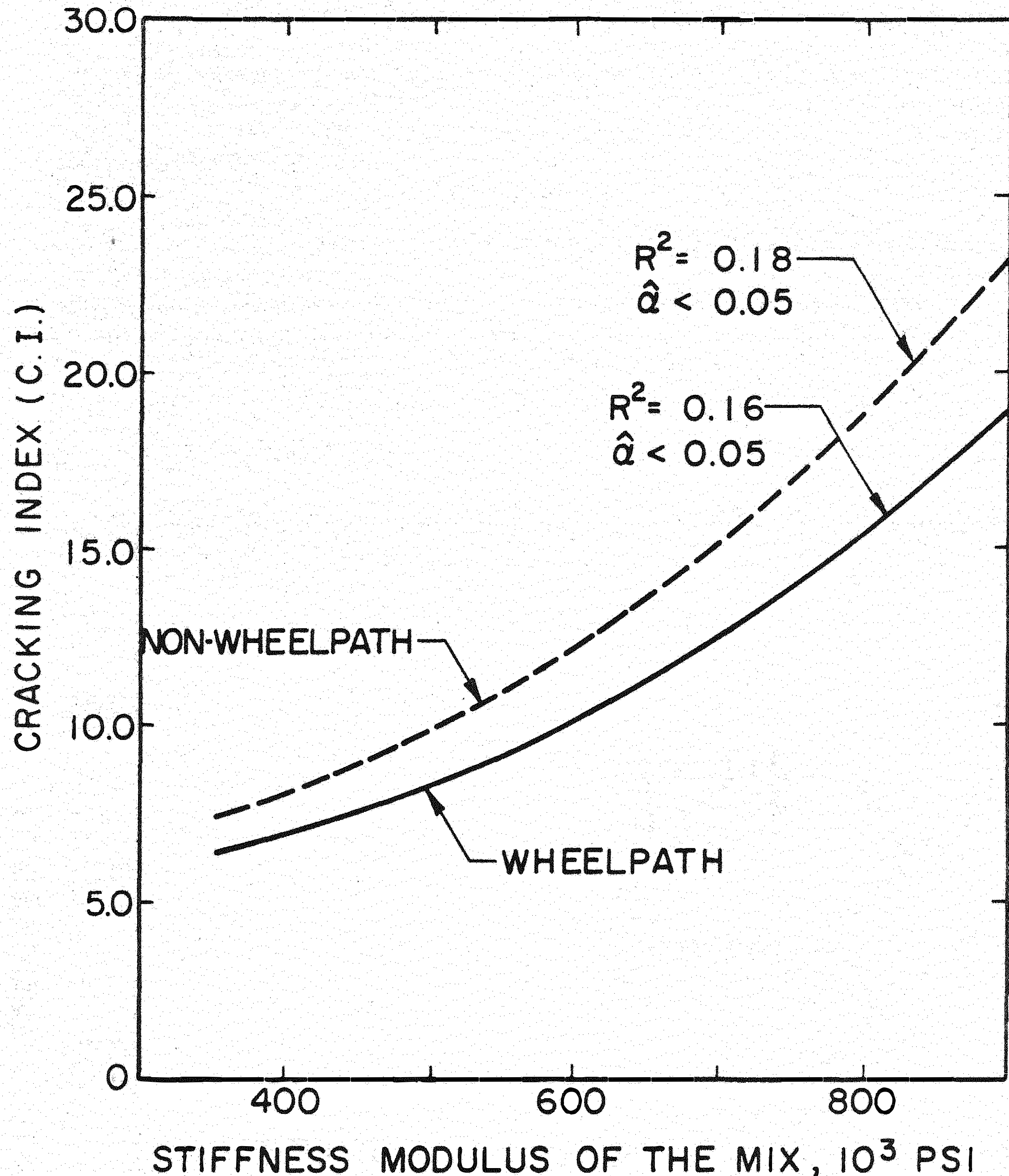

Figure 15. Relationship Between Stiffness Moduli of Field Mixtures and the Cracking Index

these temperatures. No clear trend between the penetration grade of asphalt cement and the tensile strain at failure was obtained. Test temperature influenced the effect of asphalt content on the measured tensile strains of the respective mixtures but, again, no clearly defined relationship was determined.

Although influenced by the rather erratic tensile failure strain values, some trends of the ultimate failure stiffnesses of the laboratory mixtures were obtained. As temperature decreased, the failure stiffnesses of the respective mixtures greatly increased and this increase was related to both asphalt grade and asphalt content. At all test temperatures, failure stiffnesses significantly decreased with increasing penetration grade (softer) asphalt cements in the mixtures. Again, a significant interaction between temperature and asphalt grade was observed. From these results, it appears that transverse pavement cracking in Oklahoma could be reduced by using relatively softer asphalt grades in surface course mixtures.

Stiffness Moduli of Laboratory Asphalt Cements and Mixtures

For purposes of comparision, the stiffness moduli of the respective laboratory asphalt concrete mixtures were determined on the basis of 5.0 percent asphalt content by weight of mix. This was the approximate optimum asphalt content for maximum stability for the mixtures prepared from the 91-penetration asphalt cement. Tables XIV and XV summarize the average stiffness moduli of the laboratory asphalt cements and asphalt concrete mixtures and Figs. 16 and 17 show plots of these values versus test temperature.

The tabular data and plots indicate the significant effect of

TABLE XIV

AVERAGE STIFFNESS MODULI OF LABORATORY ASPHALT CEMENTS

Asphalt Penetration at 25°C	Stiffness Modulus at Different Temperatures, Kg/cm^2				
	0°C	-10°C	-20°C	-30°C	-40°C
91	0.15	2.00	32.00	300.00	1500.00
124	0.08	0.92	10.00	150.00	800.00
160	0.04	0.50	3.50	74.00	110.00

TABLE XV

AVERAGE STIFFNESS MODULI OF LABORATORY ASPHALT MIXTURES

Asphalt Penetration at 25°C	Volume Conc. of Agg. Mix (C_v)	Stiffness Modulus at Different Temperatures, Kg/cm^2				
		0°C	-10°C	-20°C	-30°C	-40°C
91	0.859	200	1441	10,400	44,492	116,680
124	0.861	125	856	4,872	30,041	84,370
160	0.860	70	524	2,235	18,638	26,015

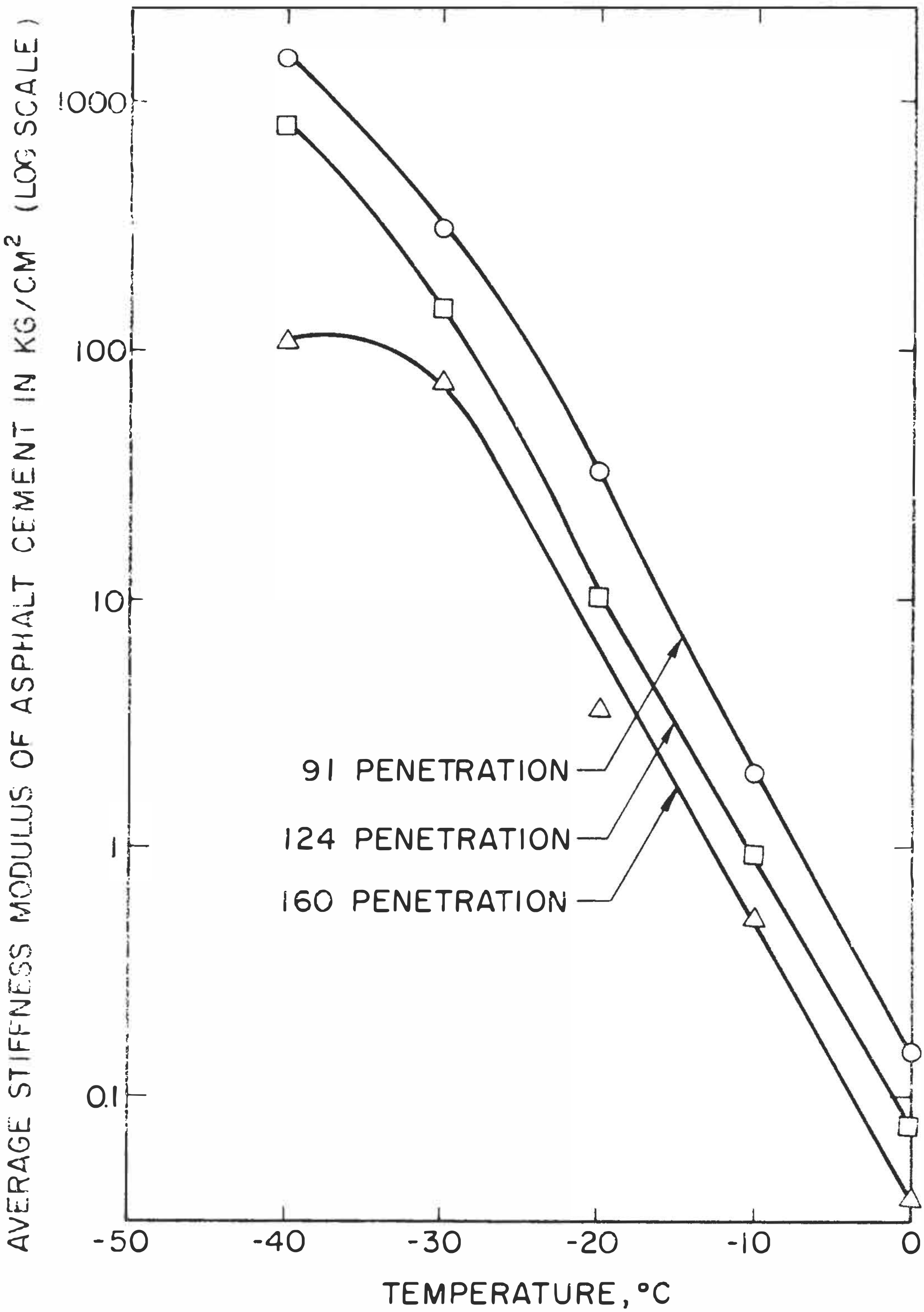

Figure 16. Stiffness Moduli of Laboratory Asphalt Cements Versus Service Temperature

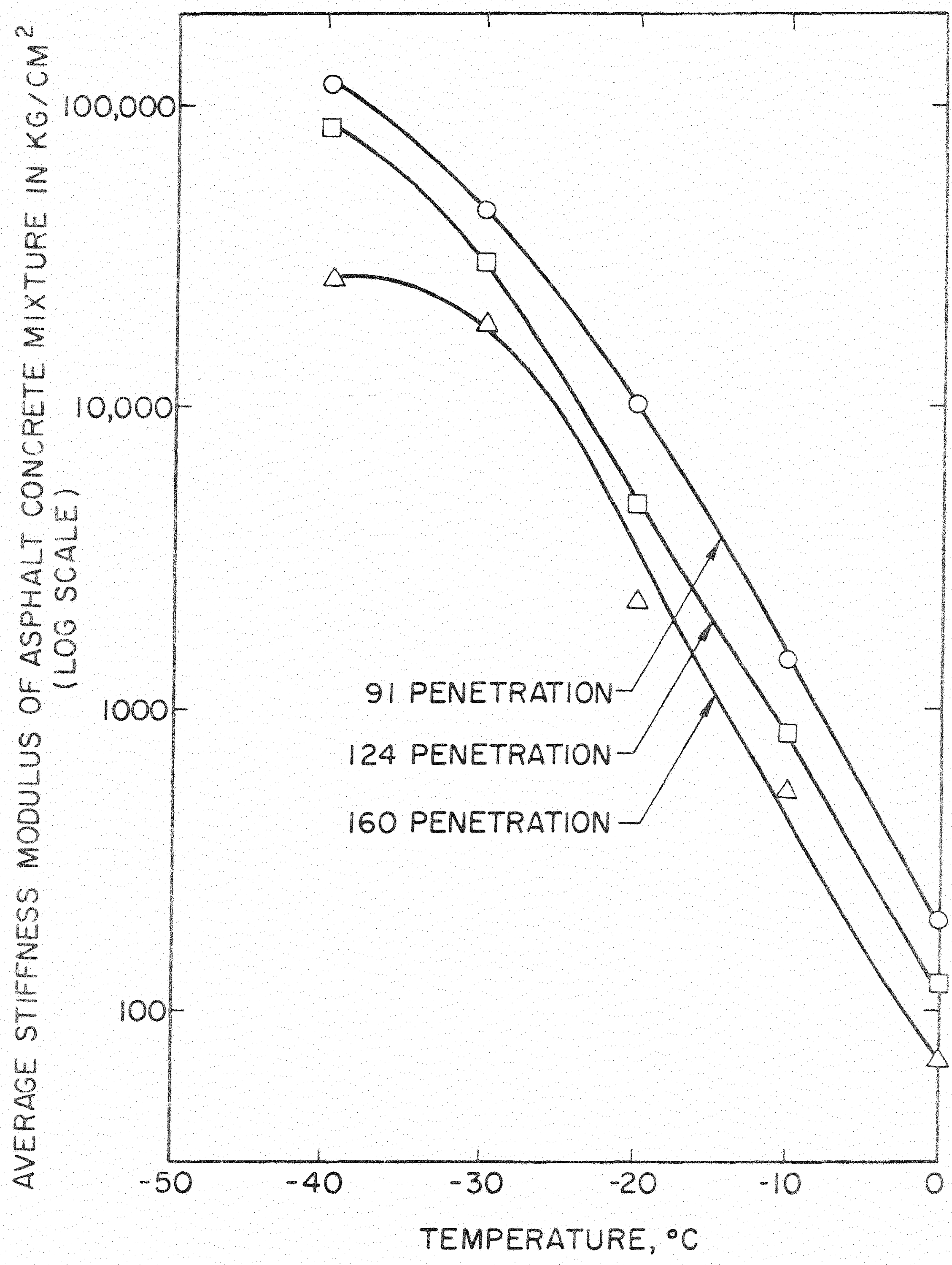

Figure 17. Stiffness Moduli of Laboratory Asphalt Concrete Mixtures Versus Service Temperature

service temperature on the performance of the asphalt cements and mixtures. The stiffness moduli of all binders and mixtures greatly increased as the temperature decreased from 0° to -30°C. The stiffness modulus at a given temperature is a function of the penetration or viscosity grade of the asphalt.

Stiffness values for the 91-penetration asphalt cement were significantly greater than those of the higher penetration asphalts and over the range of test temperatures these values increased at a rate two to four times greater than those of the softer grades. These findings point out the sensitivity of asphalt cements to low-temperature changes and indicate that low-penetration (high viscosity) asphalts will generally exhibit significantly higher stiffness moduli at low temperatures.

Previous studies have indicated that the ability of a particular paving mixture to resist transverse cracking can be predicted from the stiffness modulus of the mix at the expected minimum service temperature. From Fig. 17, the stiffness modulus of the 91-penetration asphalt mixture is approximately 17,783 kg/cm^2 (253,000 psi) at -23.3°C (-10°F). Compared with McLeod's design guide, shown in Table XVI, this value is considerably above the specified maximum safe limit to avoid transverse cracking at this temperature. Consequently, this mixture was considered to be much to stiff to prevent transverse cracking at the minimum expected service temperature in Oklahoma. Similar comparisons showed that the 124 and 160-penetration asphalt mixes could be used to alleviate the transverse cracking problem. However, these mixtures would be subject to high temperature stability problems.

Based on these results and the results of stiffness moduli determinations on twenty-six different 85-100 penetration grade samples of

TABLE XVI

MAXIMUM MIX STIFFNESS TO ELIMINATE CRACKING IN FLEXIBLE PAVEMENTS (31)

Minimum Temperature at a Pavement Depth of 2.0 in.		Stiffness Modulus	
°F	°C	Psi	Kg/cm^2
-40	-40.0	500,000	35,154
-25	-31.7	300,000	21,092
-10	-23.3	200,000	14,062
+10	-12.2	50,000	3,515

Oklahoma asphalt cements reported in Interim Report III (34), it appears that transverse cracking can be expected to occur in the hot-mix-hot-laid surface course mixtures presently specified by the Oklahoma Department of Transportation.

Conclusions

The major conclusions derived from the results of the investigation of transverse cracking are as follows:

1. Newly developed transverse cracks at the Oklahoma highway test sections were confined primarily to the surface layers and did not extend through the pavement matrix. Based on this finding and reports of other investigators of this problem, the major cause of these cracks was attributed to cold-temperature contraction of the asphalt concrete surfacing.

2. The stiffness modulus concept used by investigators in Canada and the more northern states can be adapted to Oklahoma conditions to predict the low-temperature behavior of binders and paving mixtures.

3. The tensile splitting test is a practical method of evaluating the tensile properties of both field and laboratory specimens of asphalt concrete at low temperatures. As test temperature decreased, the tensile strengths (tensile stress at failure) increased and tensile strains at failure decreased for both field core samples and laboratory specimens.

4. A satisfactory correlation was found between the tensile splitting test results and the observed amount of cracking in the field. Field core specimens taken from pavement sections with high cracking indices generally exhibited lower failure strain and higher failure stiffness values.

5. Stiffness moduli of recovered asphalts, determined at the minimum expected temperature in Oklahoma, were also correlated with the cracking indices of the pavement test sites. The stiffer or harder the asphalt cement in the pavement the greater was the degree of transverse cracking.

6. Based on the limiting stiffness approach, the 85-100 penetration grade asphalt cement specified for hot-mix-hot-laid surface course mixtures was considered too stiff a grade to avoid transverse cracking at the minimum expected service temperature in Oklahoma.

CHAPTER V

RECOMMENDATIONS

The respective investigations involved in the study and evaluation of bituminous mixes in pavement structures were diverse in that they involved different kinds of pavement performance problems. Yet, these studies were cognate to the extent that they attempted to relate the properties and characteristics of Oklahoma paving mixtures to these specific performance problems. The investigations and conclusions drawn from the results were summarized separately in the foregoing chapters and it appears desirable to similarly enumerate the major recommendations from these studies on an individual basis.

It is hoped that these recommendations are both practical and reasonable and will be considered for implementation in order to improve the performance of flexible pavements in Oklahoma. It should be kept in mind that these recommendations are not panaceas for all flexible pavement problems. The paving mixtures can and do contribute to these problems but the problems are extremely complex and many other factors such as traffic loading, environmental conditions, subgrade soil behavior and maintenance practices are all involved in the ultimate in-service performance of a pavement.

Mixes Containing Siliceous Aggregates

1. While siliceous aggregates had no detrimental effect on the

stability and cohesion of laboratory mixtures, some of these aggregates had very high stripping propensities. Thus, it is expected that surface course mixtures containing these aggregates will be prone to ravel and deteriorate rapidly at an early age. Field observations and studies of pavements using these respective aggregates in the surface mixtures should be made to determine their actual performance.

2. The stripping tendencies of some of these Oklahoma siliceous aggregates could be mitigated by using various anti-stripping additives. If raveling of these mixtures designed to improve surface skid resistance properties becomes a major problem, the stripping test techniques used in this study could be applied to evaluate the improved stripping resistance achieved with such additives.

3. The accuracy of current Oklahoma Department of Transportation laboratory procedures for determining percent density values of laboratory and field compacted specimens should be investigated. Experience in this study indicates that the "bulk impregnated specific gravity test" yields lower than actual aggregate specific gravity values. [See corresponding recommendation No. 3 under Rutting in Flexible Pavements.]

Rutting in Flexible Pavements

1. Based on the results of this study and a review of the pertinent standard specifications, it was evident that flexible pavements are presently being constructed with a build-in rutting capability. This statement applies to the surface and binder course and to the fine aggregate bituminous base course and relates to the allowable density and stability of the mixtures used in these layers. For example, the allowable density range for field compacted surface mixes in 89.3 to

93.1 percent with corresponding air void contents of 10.7 to 6.9 percent. This range of air void content is too high if a minimum amount of post-construction densification from imposed traffic loads is desired. Specific suggestions relative to more stringent specifications for minimum field densities of surface and binder mixtures and a series of recommended specification changes for the fine aggregate bituminous base (HMSA) were presented in Interim Report II (17).

2. Instability or lack of resistance to lateral displacement of the asphalt-bound materials was considered a major contributing factor to the development of ruts at many of the test sites. Thus, adequate stability of field compacted paving mixtures is absolutely essential and should be corroborated through special studies conducted by the Oklahoma Department of Transportation. The suggested studies include (a) the development of curves relating lab densities and stabilities during the standard mix design procedures for a particular job, (b) increasing the number of stability test specimens made on selected paving projects and (c) better inspection surveillance and increased numbers of density test samples and in-place measurements (nuclear density measurements) during construction to check compliance with density specifications.

3. Experience with the Oklahoma Department of Transportation's method of test for bulk impregnated specific gravity of the combined aggregate in a mix indicates that the determined values may be on the low side of what could be termed the "actual" specific gravity values of the combined aggregate. This influences the calculated theoretical maximum specific gravity values of the mixtures and results in percent density values (percent of solids by volume) of compacted laboratory specimens and field samples that are greater than their actual densities.

Thus, current laboratory mix design procedures may be partly responsible for the relatively low field densities found in this study. It is suggested that an investigation of these procedures be instituted to compare percent density values determined by the OHD-L-7 method (2) with those determined by ASTM standard method of test, D 2041.

Note: The above recommendations have been discussed in detail with ODOT personnel. The suggested revisions in standard specifications are being studied and the other suggestions have been or soon will be implemented.

Transverse Cracking in Flexible Pavements

1. Current methods of mix design are primarily concerned with the properties and behavior of paving mixtures at a maximum expected service temperature. However, the research attention given to transverse cracking in northern regions during the past fifteen years and the manifestation of this problem in Oklahoma highways indicates the need for low-temperature design supplements and modifications to alleviate transverse cracking.

2. The stiffness modulus concept or approach should be adapted to the asphalt concrete mix design procedures used in Oklahoma. Limiting stiffness values could be established and used to select appropriate binders and mixtures for specific low-temperature conditions.

3. In addition to or in lieu of the above recommendation, the tensile splitting test shown be included in the standard design procedures. "Critical" strength, strain and/or failure stiffness values at an expected minimum pavement temperature could be established. The ability of a designed paving mixture to resist transverse cracking could then be predicted by comparing its tensile splitting test results with the established limiting values.

4. Because of the high-temperature stability requirements for Oklahoma paving mixtures, various means of improving the low-temperature sensitivity of the standard 85-100 penetration grade asphalt cement presently used in surface mixes should be investigated. These studies should include the use of additives and viscosity grading specifications to alter the characteristics of the asphalt cement at low temperatures.

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