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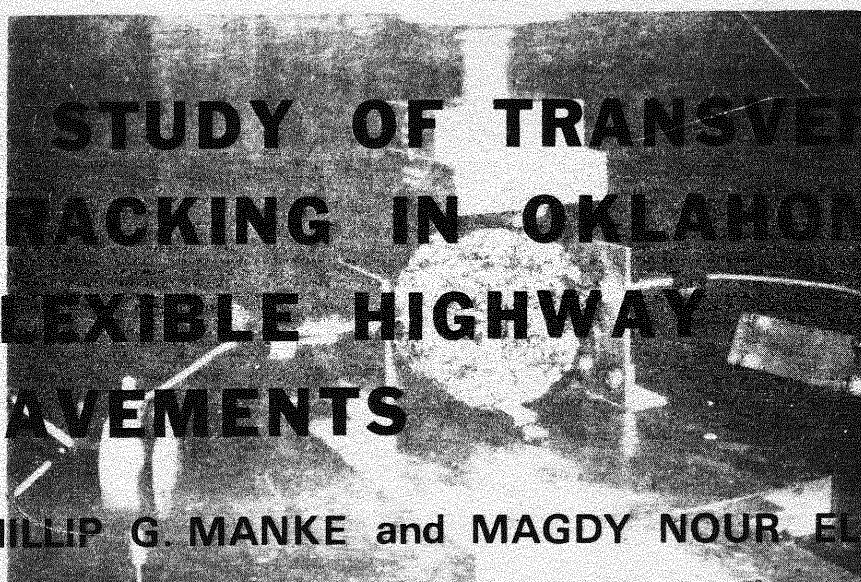
EVALUATION OF
BITUMINOUS MIXES IN PAVEMENT STRUCTURES

INTERIM REPORT V



A STUDY OF TRANSVERSE CRACKING IN OKLAHOMA FLEXIBLE HIGHWAY PAVEMENTS

PHILLIP G. MANKE and MAGDY NOUR EL DIN

SCHOOL OF CIVIL ENGINEERING
OKLAHOMA STATE UNIVERSITY
STILLWATER, OKLAHOMA

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16. Abstract

Field and laboratory studies were made to investigate the nature and extent of transverse cracks on Oklahoma flexible pavements and to determine the possible causes of this form of distress. Only the influence and contribution to this type of cracking from the bituminous components of the pavements were considered. Nine test sites on State and Interstate highway sections were studied. Examinations of field cores showed that newly developed transverse cracks did not extend through the pavement matrix and supported the concept that such cracks are caused by tensile forces developed at low temperatures.

Tensile properties of core sample specimens at 0^{o}, -5^{o} and -10^{o} were determined using a tensile splitting test. Use was made of the "stiffness modulus" concept in characterizing the behavior of the recovered asphalt binders and the paving mixtures from the cores. A satisfactory correlation was found between the tensile splitting test results and the cracking indices of the test site pavements. That is, cracking was more pronounced as the failure strains of the samples decreased and the failure stiffness increased. Stiffness moduli of the recovered asphalts also was significantly correlated with the cracking indices. The stiffer or harder the asphalt cement in a pavement the greater was the degree of transverse cracking.

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by

Phillip G. Manke
Project Director

and

Magdy Nour El Din
Research Assistant

Research Project 72-03-3
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PREFACE

Transvere cracking in flexible highway pavements is one of the serious highway performance problems in Oklahoma. A number of factors, such as asphalt and aggregate properties, construction and mix design procedures, environmental conditions, and traffic loads, can greatly affect the ability of a pavement to resist cracking. This form of cracking impairs the riding quality and shortens the life of the pavement.

This report discusses the results of a research study conducted by the School of Civil Engineering at Oklahoma State University. The main purposes of this research were to determine the nature and extent of these transverse cracks and to investigate the possible causes of this form of distress. The research was primarily concerned with the bituminous components of the pavement and their influence or contribution to the transverse cracking problem.

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P.G.M

M.S.N.

CHAPTER I

INTRODUCTION

The Problem and Its Nature

Cracking of the surface course of flexible pavements is generally considered to be a serious and extensive problem by highway agencies. Many factors, such as asphalt and aggregate properties, mix design, construction procedures, subgrade support, environmental conditions, and traffic loading, influence the ability of a pavement to resist cracking (1, 2, 3).

Transverse cracking is a significant highway performance problem in Oklahoma, in many other states primarily in the northern and western sections of the United States (2, 4, 5, 6, 7) and in many provinces in Canada (3, 8, 9, 10, 11, 12). In some cases these cracks are limited in depth and affect only the bituminous surface while others penetrate through the whole pavement structure. At an early stage, these cracks are not particularly harmful, but very poor riding qualities can result as the cracks become progressively wider and deeper. Open cracks permit the ingress of surface water which can cause stripping in the asphalt-bound materials and softening of the subgrade. In extreme cases, depressions occur at these transverse cracks due to subgrade softening and/or pumping of fine materials and secondary cracks develop parallel to the main crack.

Pavement surfaces with this kind of cracking must be repaired to prevent further deterioration and to maintain the safe smooth riding quality desired by the motoring public. The surface crack conditions may become so bad that complete re-surfacing is required long before the "design life" of the pavement is reached. Frequently, this expensive solution is unsatisfactory since these cracks tend to reflect through the new surfacing in a short time if they have not been adequately sealed prior to overlaying.

Maintaining the riding quality of flexible pavements subjected to transverse cracking can be costly. It appears that modifying current design and construction procedures to reduce or eliminate this type of pavement distress is a more desirable and economical solution to this problem.

Method and Scope of Study

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 temperatures. It seems that 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 these cracks (1, 3, 11).

The primary objective of this research was to determine the nature and extent of these cracks on Oklahoma flexible pavements and to investigate the possible causes of this form of distress. This research dealt primarily with the bituminous components of the pavement and their influence or contribution to this type of cracking.

Initially, field inspection visits were made to pavement sections affected by transverse cracking in various areas of Oklahoma. Nine test sites with various degrees of cracking were selected from these sections for further comprehensive study. Original construction data on these sites were obtained from the Research Division of the Oklahoma Department of Transportation. 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 pavement materials were recovered at various locations along newly developed cracks to study the nature and extent of these cracks in the pavement structure. Four inch (10.16 cm) diameter cores were taken at random locations in the vicinity of the cracks for further laboratory testing.

An indirect tensile-splitting apparatus was developed to evaluate the tensile properties of asphalt paving mixtures at any temperature in the laboratory. This method of testing involved loading cylindrical asphalt concrete specimens across a vertical diameter. The induced tensile stresses and strains could be calculated from recordings of load and horizontal deformation. This apparatus was employed to investigate the low-temperature tensile properties of the asphalt concrete specimens obtained from the test sites. In addition, asphalt surface course mixtures with various asphalt contents and asphalt viscosities were prepared and tested to determine the effects of asphalt content and viscosity on the tensile behavior of the mixtures at different temperatures.

The asphalt binder was recovered from the respective core specimens and tests were performed to evaluate the rheological properties of these recovered asphalt samples. Use was made of the "stiffness modulus" concept (13, 14, 15) in characterizing the behavior of the asphalt cement samples and mixtures at low temperatures. The relation of this behavior to the problem of transverse cracking of flexible pavements in Oklahoma was investigated. The Statistical Analysis System (SAS) computer program (16) was used to analyze the test results and detect the correlations between these results and the actual field behavior.

CHAPTER II

REVIEW OF PREVIOUS RESEARCH WORK

Synopsis

The non-load associated cracking of flexible pavements was first recognized in the early to mid--1930's by Rader (17). However, the problem was not serious at that time because of the low paved mileage and the small traffic volumes and loads. After World War II, traffic volumes and construction increased rapidly. Higher pavement performance standards were required and transverse cracking started to be a serious and extensive problem. During the past fifteen years, various highway agencies and investigators have devoted a great deal of effort trying to find a practical solution to the problem. In 1965, the Canadian Good Roads Association (C.G.R.A.) indicated the severity of the problem and placed it in the first priority grouping of highway research needs (18). In 1966, the Association of Asphalt Paving Technologists (A.A.P.T.) published a symposium about the same subject (19). As a result, extensive field surveying and laboratory investigations were conducted in different parts of the United States and Canada (20, 21, 22, 23, 24). Most of these studies concluded that the problem of transverse cracking could be attributed to the thermal stresses that are developed due to temperature changes, in particular, changes in the low-temperature range. One of the most significant variables of the problem was found to be the consistency characteristics of the bitumen used in the surface layer. In addition, factors of pavement age, subgrade type, and thickness of asphalt layer have been shown to be significant to the problem (1, 23, 25). In 1973, Haas presented a rather comprehensive summary of the major design approaches that have been used to analyze this low-temperature cracking (13).

Possible Causes and Major Factors

Several studies have pointed out that cracking of the bituminous surface layer is likely to occur when tensile stresses, either externally applied or internally developed , exceed the tensile strength of the material (19). In many locations having relatively cold climates, transverse cracking was developed as a result of the combination of low temperatures and the nature of the bituminous component of the pavement (1). It seems that this combination induces tensile stresses in the pavement materials through shrinkage. At a certain minimum temperature or due to sudden warming, the accumulated thermal stresses may exceed the tensile strength of the surface layer and fracture results.

Haas, et al (1) have summarized some of the factors of possible significance in low-temperature cracking of flexible pavements. These have been subdivided into external and component factors as shown in Fig. 1. It is important to realize that these different factors do not act independently but are coupled. For instance, the combination of traffic loads with a temperature drop in an asphalt concrete pavement would probably create a stress of sufficient magnitude to cause fracture.

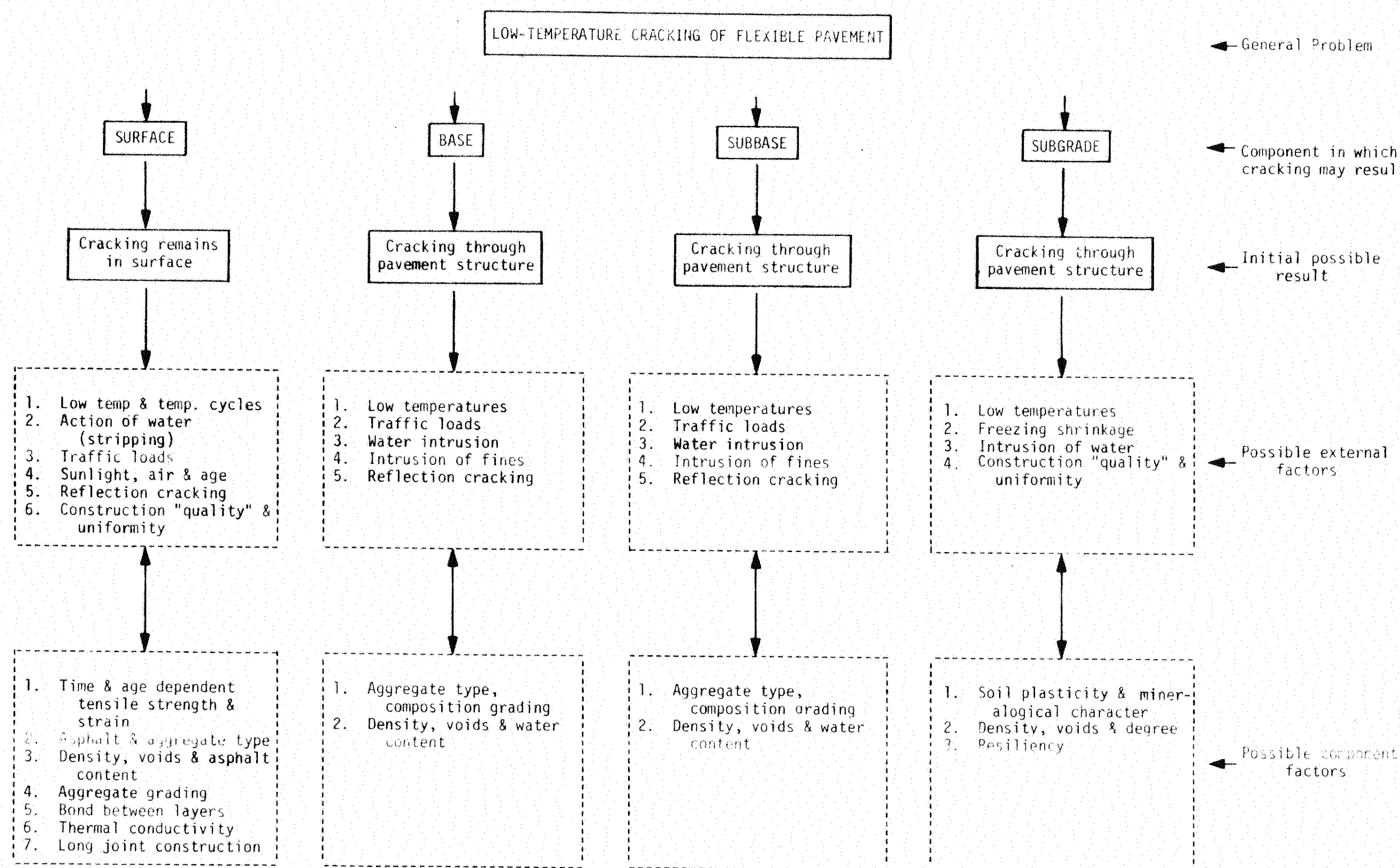


Figure 1. Factors of Possible Significance in Low-Temperature Cracking of Flexible Pavements (1)

Cracking Mechanism

Laboratory investigations conducted on asphalt cements and mixtures have shown that they lose some of their plasticity at low temperatures and behave essentially in an elastic manner. At the same time, the asphalt layer tends to contract but is partially or wholly restrained. The combined effect of both responses is the development of the fracture tensile stresses. This crack mechanism was substantiated with actual field records which indicated that fracture occurred when pavement surface temperatures reached a minimum (10, 26, 27, 28, 29). Because of pavement geometrics, the principal axis of contraction is in the longitudinal direction and, hence, the majority of thermal cracks are to be expected in the transverse direction.

It seems that these thermal cracks usually start in the form of micro-cracks at the surface of the asphalt layer (1). It was also found that such cracks subsequently open with time and propagate to the full depth of the pavement structure after one or more temperature cycles (30). Some studies concluded that cracks occurring during the first season relieve the developed stresses, and that additional cracking in succeeding years would occur at a reduced rate or frequency (19).

Field Investigations

Field investigations devoted to this problem can be divided into two broad approaches. The first was mainly oriented towards surveying, sampling and testing the existing cracked pavements in order to document the extent of the problem and attempt to correlate certain factors with the frequency of cracking (1, 3). The second approach was primarily concerned with studying the effect of different materials and structural variables on the actual field performance of full-scale pavement sections (27, 29). Field testing programs included mapping the different cracking patterns and coring and sampling of base and subgrade materials. Cracking frequency diagrams were prepared to help in selecting specific areas for further detailed examination. This cracking survey also included the number of part, half, full and multiple transverse cracks occurring in each 500 ft interval of the pavement. Fig. 2 represents a diagramatic illustration of the previous four categories of cracks.

To develop a measure of cracking severity, a cracking index was developed by the Ontaria Department of Transportation (22). This index is defined in the following equation:

$$C.I. = N_m + N_f + 1/2\ N_h \quad \text{(2.1)}$$

Where C.I. = Cracking index

$$\left.\begin{array}{l} N_m = \text{Number of multiple cracks} \\ N_f = \text{Number of full cracks} \\ N_h = \text{Number of half cracks} \end{array}\right\} \text{per 500 ft of 2-lane pavement}$$

It was felt that smaller transverse cracks usually occur subsequent to the formation of half or full cracks and, therefore, these smaller cracks were disregarded in the calculation of the cracking index (22).

Some initial field studies indicated that variables such as asphalt type, grade and source, as well as age and thickness of pavement, showed a significant correlation with the degree of cracking of pavement sections (9, 19, 31). Consequently, several full-scale experimental sections were constructed in various areas in Canada to evaluate the effect of these variables, particularly those concerning asphalt type and grade (28, 29, 32, 33). Air and subsurface temperatures were continuously

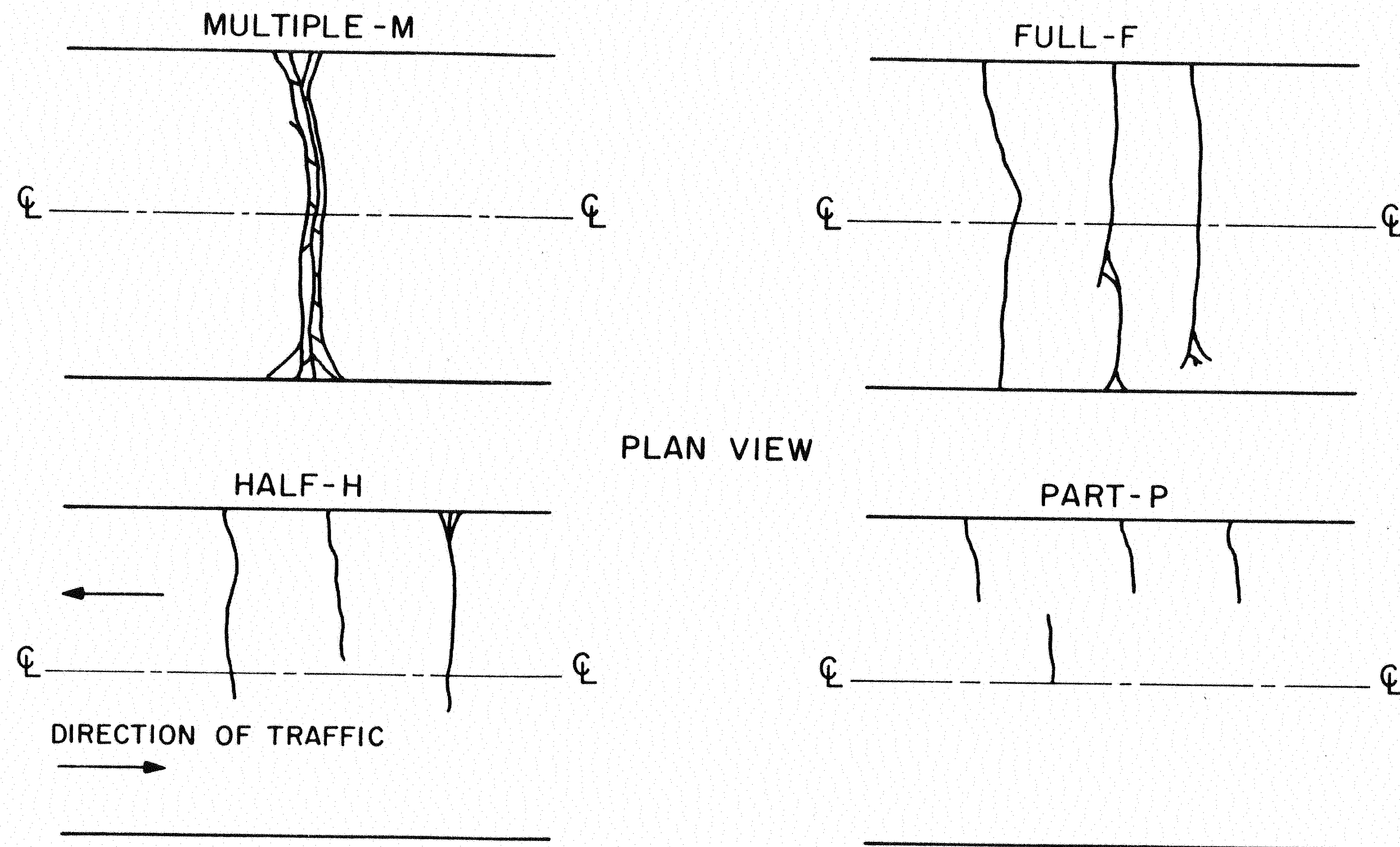


Figure 2. Different Types of Transverse Cracks (22)

recorded at these sections. Periodic crack surveys were made to detect crack initiation and to closely follow the propagation and development of these cracks. In addition, the structural capacity of these experimental sections was evaluated by field test procedures (10, 26, 27, 28). The analysis of the performance data on these sections again showed a significant correlation between the asphalt source, type and grade and the observed degree of cracking. However it was emphasized that low-temperature cracking appeared to be a complex phenomenon with many factors involved and that other variables could be highly important in certain situations (1).

Material Characterization

As previously concluded, the bituminous component of the pavement was a major variable in the low-temperature cracking problem. This result was strong enough to guide the previous research efforts towards finding new methods of characterizing the behavior of asphalt cements and mixtures at low temperatures. Various investigations have pointed out that one of the most satisfactory and applicable approaches is that of the stiffness concept (13, 14, 15). Other research studies recommended the use of the tensile splitting test to evaluate the low-temperature tensile fracture strengths and strains of asphalt concrete materials, either secured from the field or freshly prepared (34, 35, 36). Another important parameter in previous analysis and design approaches was the coefficient of thermal contraction of the paving mixture. Several investigations have been performed to determine this coefficient, some of which indicated that the coefficient is not linear over the entire low-temperature range. Phang (37) has conducted a comprehensive set of tests to evaluate the contraction coefficient and he reported values for field samples ranging between 0.95 x 10^{-5}/F° and 1.81 x 10^{-5}/F°, with an average value of 1.5 x 10^{-5}/F° in the temperature range -20° to + 30°F.

Available Design Approaches

According to Haas (13), several major alternative design approaches to low-temperatures shrinkage cracking have been developed. However, he indicated that some of these approaches are highly empirical and do not make provisions for estimates of error. Haas also pointed out that these approaches can be temporarily used until the more fundamental procedures have been fully developed and validated. These approaches may be grouped into four categories:

1. Specification adjustment.
2. Limiting stiffness.
3. Fracture temperature.
4. Statistical correlations to observed cracking.

Specification Adjustment

The behavior of asphalt concrete mixtures at relatively high temperatures has been the main interest of the paving technologists for many years. Thus,specifications for controlling asphaltic paving mixtures were only concerned with stability at 140°F and density and voids at 77°F and little interest was evidenced in controlling the behavior of asphaltic mixtures at low temperatures. Unfortunately, the importances of stability at high temperature many have been overemphasized. In many cases, high stability has been attained at the expense of plasticity by using low-penetration asphalt cements and, as a result, these mixtures were more susceptible to cracking at low temperature. This has suggested a new approach for design criteria. Several agencies have adjusted their specifications to require softer (higher penetration) grades of asphalt, while others retained their penetration grades but have specified a fairly high minimum voscosity requirement at either 140° or 275°F (1).

It seems that some progress has been achieved in some cold regions with using only softer grades of asphalt cement (1, 11, 23). However, Haas (1, 13) has stated that, if softer asphalts are used, there can be long-term effects on other aspects of pavement performance, such as rutting and fatigue, that should be evaluated.

Limiting Stiffness

The concept of limiting stiffness in the asphalt and/or mix has been used in different studies to develop an asphalt selection guide for various low temperatures. It was thought that cracking could be eliminated, if the stiffness of the mix did not exceed a certain maximum at the minimum service temperature (23). A rather detailed discussion of this design approach was the subject of Interim Report III (15).

Fracture Temperature Prediction

It has been pointed out that the postulated mechanisms of cracking were based upon the concept of having induced thermal stresses that exceeded the tensile fracture strength of the pavement. The estimate of fracture temperature depends on calculating these stresses due to a drop in temperature and comparing them with the tensile strength of a given layer. When the tensile strength is exceeded by the accumulated stresses, fracture should occur as shown in Fig. 3. The tensile strength of asphalt concrete mixtures may be determined either by direct experiment, such as the tensile splitting test, or by indirect estimation methods (1).

Hills and Brien (38) have originally proposed an approximate method to calculate the thermal stresses in a long completely restrained strip using the following equation:

$$\sigma_x(T) = \alpha \sum_{T_\circ}^{T_f} S(\Delta T) \cdot \Delta T \quad \text{.................} \quad (2.2)$$

where $\sigma_x(T)$ = accumulated thermal stress for a particular cooling rate, T.

α = average thermal contraction coefficient over the temperature drop, $T_\circ - T_f$.

$T_\circ, T_f$ = initial and final temperature.

$S(\Delta T)$ = stiffness at the midpoint of discret temperature intervals ΔT over the range of $T_\circ$ to T_f, using a loading time corresponding to the time interval for the ΔT change.

Haas and Topper (30) have extended this approach to include both temperature and stiffness gradients through the depth of the asphalt layer. The method they developed was used to calculate thermally induced stresses in several previous efforts (21, 33, 39) and to predict the probable fracture temperature. Haas (13) pointed out that some limitations should be kept in mind while using the fracture-temperature prediction approach, and Shahin (40) has indicated that a single fracture temperature can not be considered a satisfactory criterion for design because of the variation of the asphalt concrete properties over the entire road length.

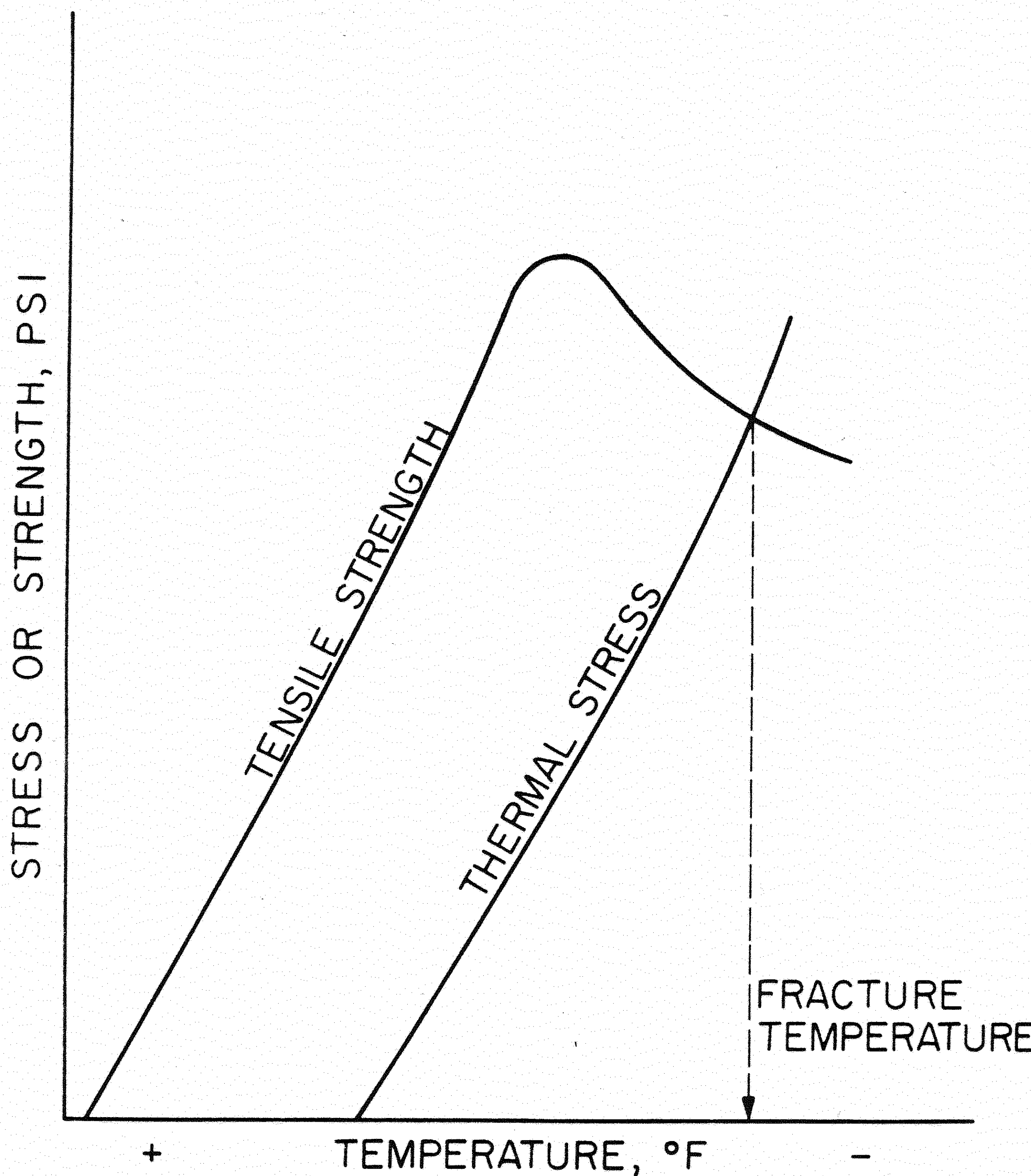


Figure 3. Schematic Diagram of the Fracture Temperature Concept

Statistical Correlations to Observed Cracking

Observations and data collected from various research studies have indicated that certain variables were significantly related to the observed degree of cracking. These variables have been used by several investigators in developing mathematical models to estimate the low-temperature shrinkage cracking frequency of future pavements, on the basis of past experience. Fromm and Phang (22) have developed equations for a cracking index for Ontario conditions based upon a stepwise and linear multiple regression techniques. Likewise Hajek and Haas (41) have constructed a predictive model which estimates the cracking frequency in terms of a cracking index for various ages in the life of the pavement. This model may be defined as follows:

$$C.I. = f(s, t, a, m, d) \qquad (2.3)$$

where C.I. = cracking index.

s = stiffness (kg/cm^2) of the original asphalt cement, determined by McLeod's method (23) for a loading time of 20,000 seconds and at a winter design temperature m.

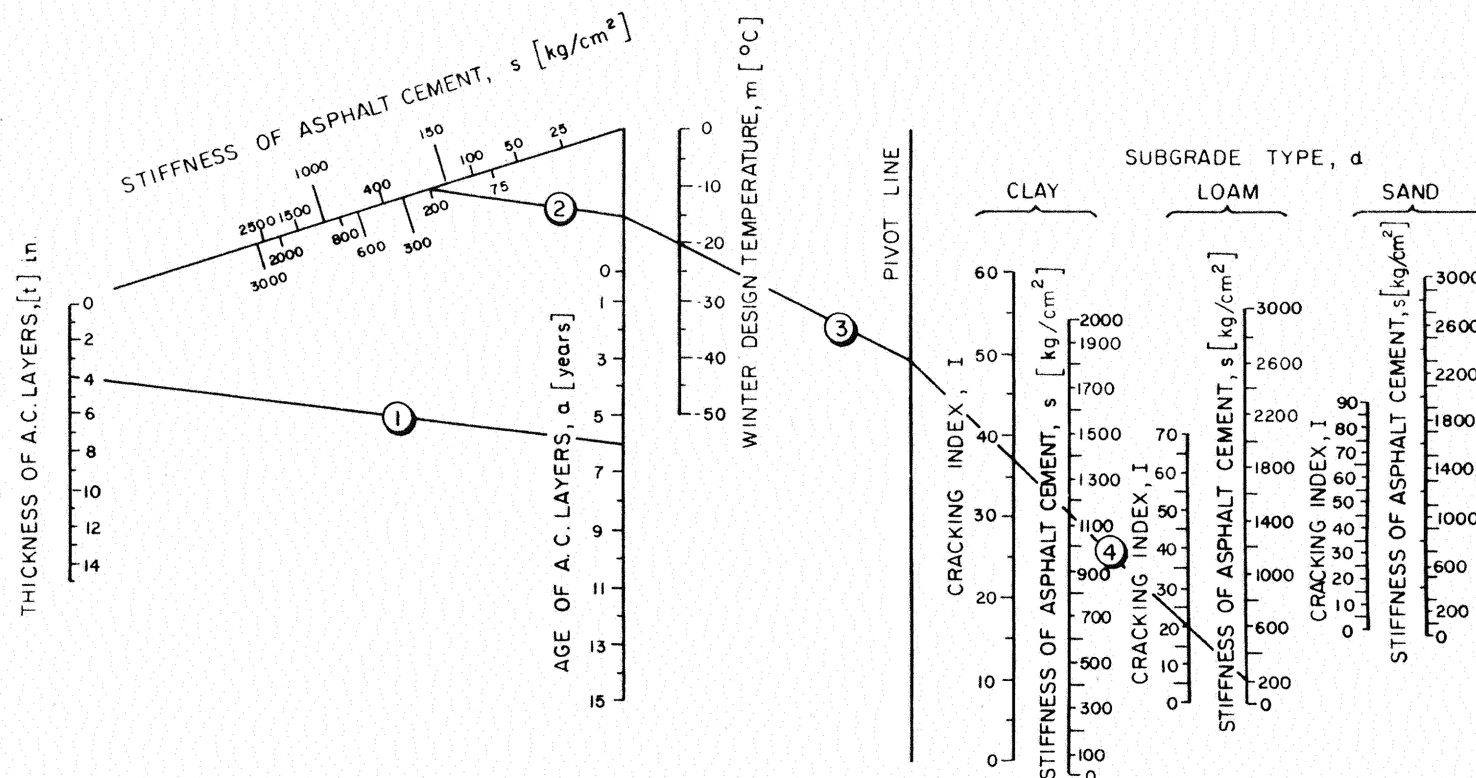
t = combined thickness of the asphalt layers (in.).

a = age of pavement (yrs.).

m = winter design temperature (C°).

d = subgrade type code, dimensionless [d = 5 (sand); d = 3 (loam); d = 2 (clay)].

A nomographic solution for this model has been developed and is shown in Fig. 4. Haas (13) showed that the model was evaluated in terms of its statistical significance, rational behavior and relation to the observed data. The results suggested that the model could be used for initial design purposes with a considerable degree of confidence.



EXAMPLE:

thickness = 4 inches
age = 6 years
stiffness of original asphalt cement = 200 kg/cm²*
winter design temperature = -20 °C
subgrade type = loam
*for temp. = m and time = 20000 sec.

NOTE:

A) lines ① and ② are parallels
B) in step ④ select scales for the appropriate subgrade type

RESULT: cracking index = 20 at 6 years

Figure 4. Nomograph for Predicting Low-Temperature Cracking Frequency of Asphalt Pavements (13)

CHAPTER III

EXPERIMENTAL DESIGN

The design of an experiment can generally be defined as a complete sequence of steps taken to insure that appropriate data will be obtained for objective analysis (42). This concept was utilized in planning and conducting the experimental field and laboratory programs to provide a maximum amount of information relevant to the transverse cracking problem.

The objective of the field testing program was to investigate the possible causes of cracking and the factors that caused contrasting degrees of cracking to occur in particular highway pavements. Various highway sections exhibiting different types of transverse cracking were visited. Some of these sections were suggested by the Research and Development Division of Oklahoma Department of Transportation and others were located by research personnel during field trips to different locations in Oklahoma. Preliminary surveys of these sections were made to: 1) check the sight distance available to on-coming traffic, 2) visually examine the degree, nature and extent of existing cracks, 3) locate recently developed cracks for further study, 4) make a general rating of pavement surface condition, and 5) study the geometric design characteristics of the pavement sections.

A great deal of consideration was given to the vertical and horizontal alignment at these sections to assure that the safety of research personnel could be maintained during further field operations. Tentative selection of possible test sites depended mainly on available safe sight distances. Any site on a horizontal or vertical curve was disregarded and, where possible, another location on a tangent section in the same vicinity was selected. Suitable field locations were marked and identified as possible test sites for the experimental field study.

Selection of the test sites was based on both the results of the initial field survey and the planned time schedule. Consideration was given to the selection of test sites where a relatively wide variation in degrees of cracking had occurred in order to possibly identify and contrast the contributing factors existent at these sites. Also, transverse cracking was observed at pavement sections on both the high-quality interstate and lower quality state highway systems, and test sites from both types of facilities were included in the study. In order to investigate the effect of aging on the observed degree of cracking, pavements of various ages were considered in the final selection of the test sites.

To keep the amount of experimental work within practical time limits, a total of nine test sites was finally selected for detailed investigation. Permanent identification of these sites was made by attaching a 10.0 in. x 11.0 in. (25.4 cm x 27.94 cm) red painted metal sheet to the right of way fence at each test site. These markers were located at a measured odometer distance from the nearest intersection, bridge, or the boundary line of the county in which the test sites were located. At each test site, a 500 ft length of pavement which satisfied the safe sight distance requirements was chosen for detailed crack surveying, counting and coring. A Rolatape (Model 200) was used to measure the 500 ft length of the pavement and the beginning and ending points were marked with a yellow paint stripe along the shoulders.

The selected test sites included four sections on State Highway No. 177 where various degrees of cracking were observed. The other five sites were on sections of Interstate 35 and Interstate 40. Two of these interstate sites had almost no cracking and were chosen for comparison. Table XI in Appendix A shows the exact locations and the construction and maintenance dates for these sites.

Field Testing Program

The field study started with mapping and counting the cracking patterns within the chosen 500 ft lengths of pavement. These data were used to develop a cracking index for the sections as a measure of cracking severity. In addition, newly developed cracks, i.e., extremely narrow transverse cracks that did not extend the full width of the pavement, were selected at various sites during the crack survey. Pavement cores spanning these narrow cracks were obtained in an attempt to ascertain the mechanism of transverse cracking.

Another part of the field study consisted of securing pavement core samples for further laboratory testing. A previous study (43) showed that traffic loads tended to densify the asphalt concrete paving materials. Consequently, it was decided that any evaluation of the pavement core samples should consider their location with respect to traffic wheel paths and the field cores were classified as "wheelpath" and "non-wheel-path" samples. It was also planned to evaluate the tensile properties of the field samples at three low temperatures. To accomplish this, eighteen field core specimens were required at each test site. At test sites where almost no cracking was observed, core specimens were taken from nine wheelpath and nine non-wheelpath locations within the chosen 500 ft length of pavement. At other test sites, randomization principles were used to choose three full cracks from those previously counted during crack surveying of the individual test sites. At appropriate off-set distances from the edges of these cracks, six core specimens were obtained as illustrated in Fig. 5. These cores were also identified as wheelpath and non-wheelpath samples.

Laboratory Testing Program

The laboratory testing program was designed largely on the basis of the results of the study of the preliminary pavement cores. As previously mentioned, these were obtained by coring across very narrow transverse cracks. Examination revealed that in a majority of cases these "beginning" cracks did not extend through the pavement matrix. Apparently, the cracks had originated at the surface and had propagated or extended to a limited depth in the underlying layers (Fig. 6a). In a few core specimens, these narrow cracks extended completely through the asphalt paving layers (Fig. 6b). It is possible that these full-depth cracks are also due to thermally induced stresses in the pavement and have had the necessary time and ambient conditions to propagate the full pavement depth. However, there is an alternate possibility that these cracks first occurred at the pavement-subgrade interface and resulted from load induced stresses and/or subgrade soil aberrations.

Based on the hypothesis that the transverse cracks are caused by tensile forces developed in the pavement surface and that these forces are due to the low-temperature response of the asphalt paving mix, the research approach was directed towards evaluating the low-temperature

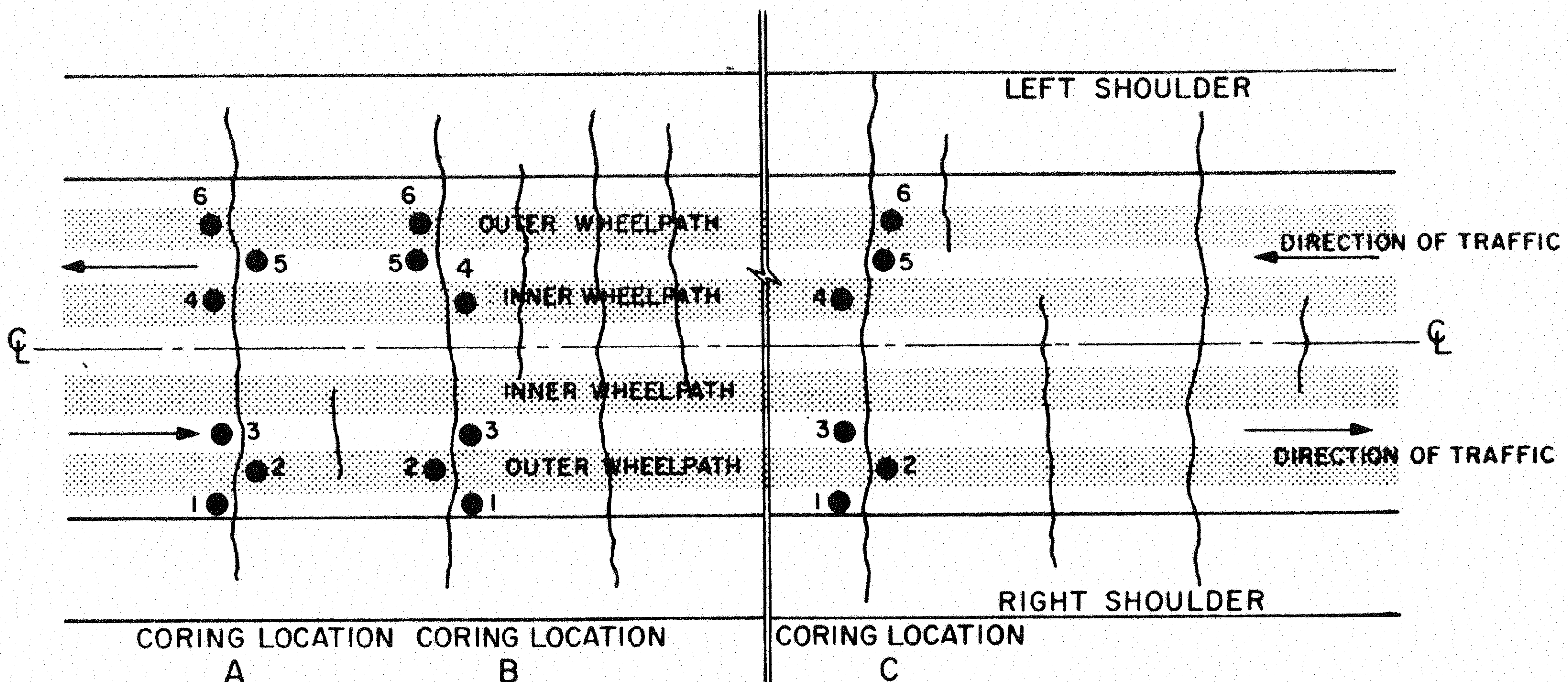
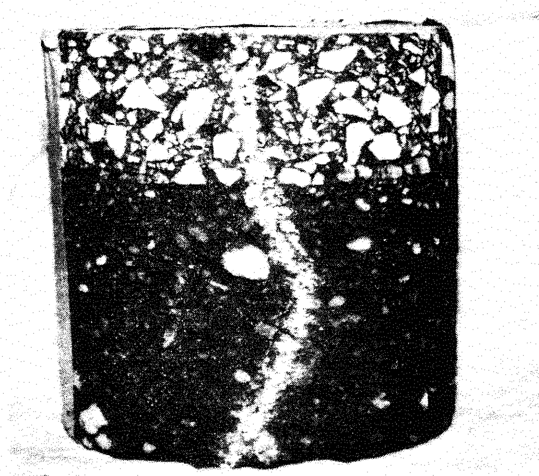


Figure 5. A General Layout of Coring Plan for a 2-Lane Test Site

(a)



(b)

Figure 6. Extension of Transverse Cracks in the Pavement Matrix

behavior of the asphalt materials and mixtures and correlating this behavior with the actual field performance data, i.e., the cracking index of the pavement section.

Evaluation of Field Samples

The first part of the laboratory testing program was an evaluation of the properties of the asphalt concrete surface course of each test site. The upper layer of each core specimen was cut and the thickness and diameter were measured. Bulk specific gravity values of the wheel-path and non-wheelpath paving mixtures were determined by averaging the values for three representative samples. To evaluate the tensile properties of field samples over a low-temperature range, three temperatures, 0°, -5°, and 10°C, were chosen for the tests. A relatively simple and practical tensile splitting apparatus (Chapter IV) was used to determine the tensile properties of these field specimens at the various temperature levels.

The design of the experiment can be defined as a "split-plot design" (44) where field samples represented experimental units obtained from nine test sites (blocks). Experimental units secured from each test site were divided according to their location into two "main plots", wheelpath and non-wheelpath. Each main plot was subdivided into three subplots to which the aforementioned temperature levels were randomly applied. To increase the precision of the experiment, each temperature level (subplot treatment) was applied to three samples (replicates).

A modified Rice's specific gravity test was used to determine the maximum specific gravity of the field samples . These test samples were those previously chosen for bulk specific gravity determinations and the test results were used to calculate the percent density and air void content of the wheelpath and non-wheelpath materials.

The asphalt cement binders were extracted from the pavement materials and the average asphalt content of each surface mix was determined. These asphalt binders were recovered from the extraction solutions for further laboratory testing. Three recovered asphalt samples from each test site were tested to determine their rheological properties. These properties were used in calculating the stiffness moduli of the various asphalt materials and mixtures, according to McLeod's approach. These stiffness moduli were calculated at a service temperature of -10°F (-23.3°C). The choice of this temperature was based upon the climatological data of Oklahoma and the pavement temperature data reported in another research study (27).

The laboratory test results and the calculated stiffness values were correlated with the actual field performance data of individual test sites as measured by their cracking indices. However, it is important to note that the chosen test sites had not been sampled just at the point of incipient cracking. Apparently, some had cracked years before and considerable aging of the asphalt binders had taken place since the initial cracking. Other pavements had few cracks and their properties were still some distance from the critical point. Thus, the best that could be hoped for was a considerable scattering of values when correlated with the amount of cracking that had occurred.

Evaluation of Laboratory Materials and Mixtures

The experimental approach of characterizing the behavior of asphalt materials and mixtures at low temperatures was extended to investigate and/or predict the performance of fresh asphalt materials and mixtures.

Eighty-one compacted specimens of a Type C surface course mixture (45) containing various asphalt contents and different penetration grades of asphalt cement were prepared and tested using the tensile splitting apparatus. This was done to study the effect of three factors, temperature, asphalt content, and asphalt penetration grade, on the tensile properties of the asphalt concrete mixtures. Three levels of each factor were employed in this study to allow for useful comparisons. These 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.

Based on the statistical definition of "design of experiment", this was considered as a "3 x 3 x 3 factorial experiment" where treatments were the various possible combinations of factors and factor levels. Three specimens (replicates) per each treatment were tested to increase the precision of treatment comparisons.

In Interim Report III (15), the stiffness moduli of the asphalt binders and mixtures were calculated at a -40°F service temperature. In this study, however, the method was extended to calculate the stiffness moduli at various low temperatures and compare the calculated values to the limiting stiffness values reported in the literature for the same conditions of temperature and rate of loading. Based on the results of these comparisons, it was hoped that a quantitative selection guide for asphalt binders and/or mixtures can be established as an aid in preventing low-temperature transverse cracking.

CHAPTER IV

DEVELOPMENT OF THE TENSILE SPLITTING TEST EQUIPMENT

The tensile splitting test was originally developed to evaluate the tensile strength of concrete and mortar materials (46, 47). In recent years, the use of this test was extended to include cement-treated gravel, lime-soil mixtures and asphalt-stabilized materials (24, 34, 48, 49). This test involves loading a cylindrical specimen with a compressive load distributed along two opposite generators. This results in a failure that usually occurs by splitting along the diametral plane as shown in Fig. 7.

The tensile splitting test used in the experimental phase of the study provided a relatively simple means for determining the tensile properties of asphalt concrete specimens at low temperature. It was hoped that this test procedure would be useful in investigating the ability of the common asphalt paving materials in Oklahoma to resist shrinkage cracking at low temperatures.

Theory of the Test

The theoretical concept of the tensile splitting test is based on the theory of elasticity. Assuming plane stress and considering a circular disk subjected to concentrated loads on the diameter as shown in Fig. 8, the general equations that express the tensile, compressive and shear stress on an element within the disk, are as follows:

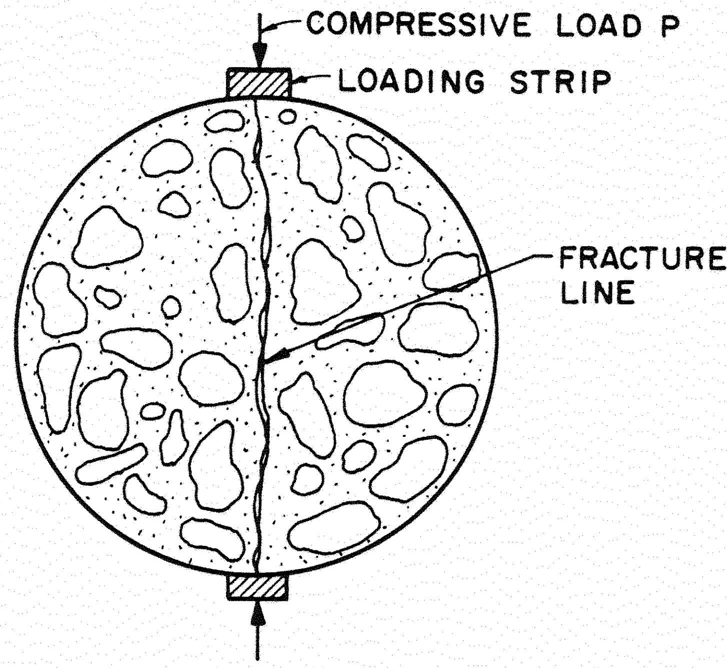


Figure 7. The Indirect Tensile Splitting Test

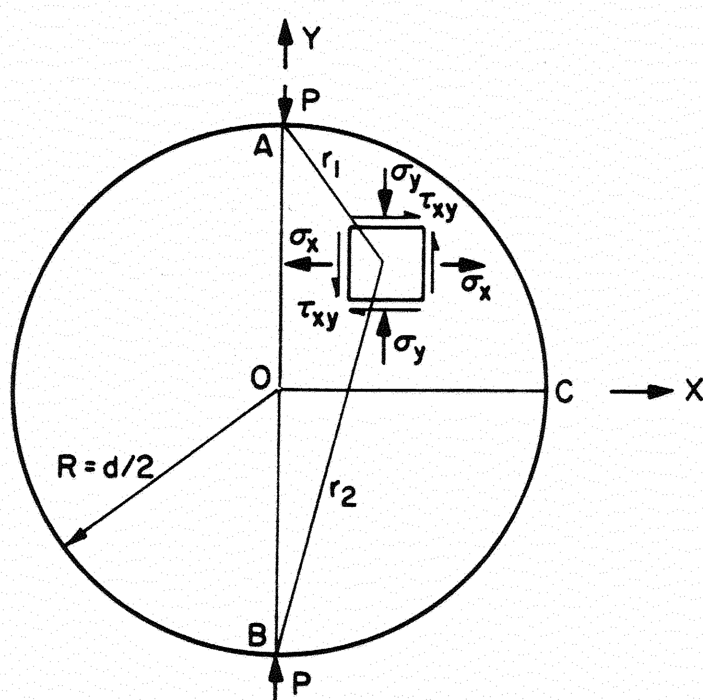


Figure 8. Diagram Showing the Loading Disk and the Resultant Stress Conditions (46)

$$\sigma_x = \frac{-2p}{\pi t}\left[\frac{(R-y)x^2}{r_1^4} + \frac{(R+y)x^2}{r_2^4} - \frac{1}{d}\right] \text{.........} (4.1)$$

$$\sigma_y = \frac{-2p}{\pi t}\left[\frac{(R-y)^3}{r_1^4} + \frac{(R+y)^3}{r_2^4} - \frac{1}{d}\right] \text{.........} (4.2)$$

$$\tau_{xy} = \frac{2p}{\pi t}\left[\frac{(R-y)^2x}{r_1^4} - \frac{(R+y)^2}{r_2^4}\right] \text{...............} (4.3)$$

Where σ_x, σ_y, τ_{xy} = stress components with respect to rectangular co-ordinates.

x, y = rectangular co-ordinates.

p = load applied to specimen.

t = thickness of cylindrical specimen.

d = diameter of cylindrical specimen.

R = radius of cylindrical specimen.

r_1, r_2 = location co-ordinates.

Considering points between 0 and C on the horizontal x-axis where $y = 0$ and $r_1 = r_2 = \sqrt{x^2 + R^2}$, both σ_x and σ_y vanish at the circumference and reach maximum values at the center. These maximum stresses can be written as follows:

$$\sigma_x = \frac{2P}{\pi td} \text{.....................................} (4.4)$$

$$\sigma_y = \frac{-6P}{\pi td} \text{.....................................} (4.5)$$

This indicates that the material tested should have a compressive strength at least three times its tensile strength to assure a tensile failure.

Along the y - axis, where $x = 0$, $r_1 = R - y$, and $r_2 = R + y$, the stress equations reduce to :

$$\sigma_x = \frac{2P}{\pi td} \text{.......................................} (4.6)$$

$$\sigma_y = \frac{-2P}{\pi t}\left[\frac{2}{d-2y} + \frac{2}{d+2y} - \frac{1}{d}\right] \ldots\ldots\ldots\ldots\ldots\ldots (4.7)$$

$$T_{XY} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.8)$$

The distribution of stresses along a horizontal and a vertical diameter, as calculated from the previous equations, are shown in Fig. 9 and Fig. 10.

As can be seen in Fig. 10, the horizontal tensile stress σ_x, along the vertical plane has a constant value of $2P/\pi td$ and the vertical compressive stress, σ_y, has a minimum value of $-6P/\pi td$ at the center and a maximum value of infinity at the load points. With a line load, it seems that the specimen is likely to fail near the load points due to the high compressive stresses. However, photoelastic studies (46) have shown that these compressive stresses can be greatly reduced by using a distributed load applied through a short loading strip. These loading strips were found to be sufficient to retard the compressive failure at the load points so that the specimen fails due to the tensile stress at the center (47).

The total tensile strain at failure for a 4.0 in. (10.16 cm) diameter specimen with a 0.5 in. (1.27 cm) curved loading strip can be determined as follows (50):

$$\varepsilon_{TF} = X_{TF}\left[\frac{0.1185\nu + 0.03896}{0.2494\nu + 0.06730}\right] \ldots\ldots\ldots\ldots\ldots (4.9)$$

where ε_{TF} = total tensile strain at failure, in./in.

X_{TF} = total horizontal deformation at failure, in.

ν = Poisson's ration of asphalt concrete material.

Previous studies have shown that Poisson's ration of asphalt concrete materials varies between 0.25 and 0.35 with an average value of 0.3 (50).

Substituting this value in equation 4.9, the total strain at

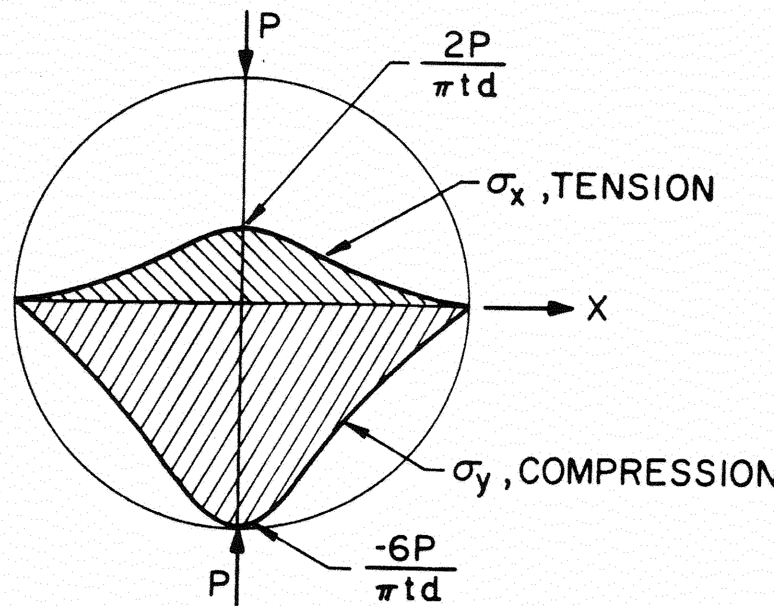


Figure 9. Stress Distribution on X-Axis (49)

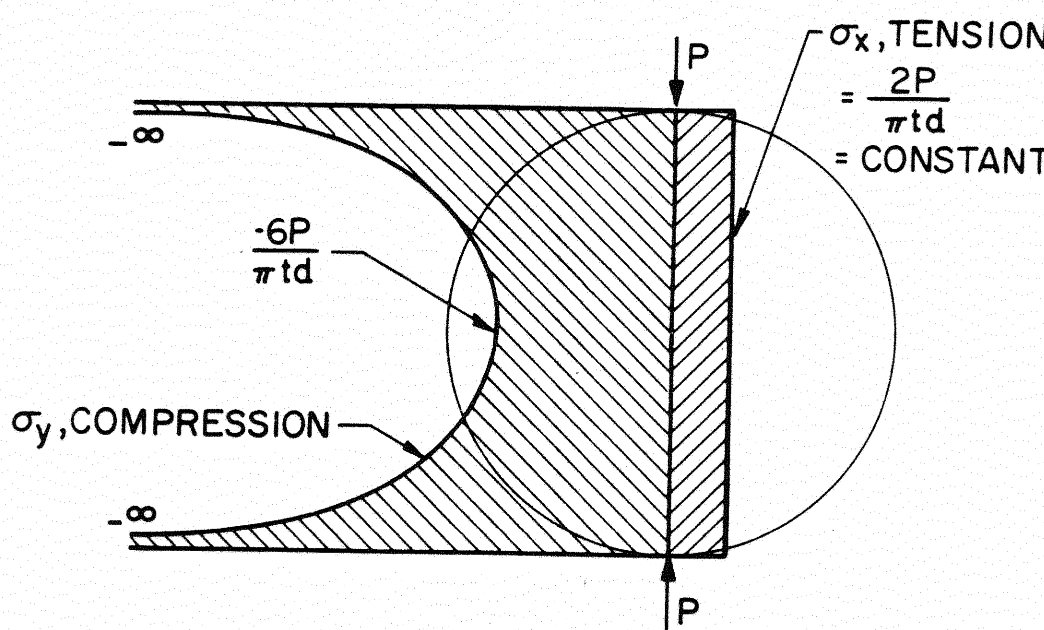


Figure 10. Stress Distribution on Y-Axis (49)

failure can be expressed as:

$$\varepsilon_{TF} = 0.524\ X_{TF} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.10)$$

Deviations of Test From Ideal Conditions

The previous theoretical analysis was based on an idealized case of an elastic, isotropic and homogeneous material. Actually, the test deviates from the assumed ideal condition. However, some of these deviations were thought to have a minor effect on the results obtained from the test (51). For instance, it was indicated that the random distribution of aggregate particles in an asphalt concrete specimen tends to minimize the effect of heterogeneity. Also, the assumption of plane stress was considered to be valid for the 2.0 in. (5.08 cm) thick specimens, since they are relatively thin. Also, the behavior of asphalt concrete materials at low temperature is essentially elastic in nature, therefore, the assumption of elasticity of the material may be reasonable under this condition.

One of the main test deviations is the loading method. Actually, the line load is distributed over an area because of the practice of applying the load through a loading strip. Studies concerning the effects of a loading strip on stress distribution have shown that the magnitude of the vertical compressive stresses is considerably reduced and that the magnitude of the horizontal tensile stresses is generally unchanged near the center of the specimen. However, the tensile stress changed to compression directly beneath the loading strips (47). Therefore, it may be concluded that the effect of the loading strip is quite localized and that the specimen will ordinarily fail in the central portion due to tensile stress.

Test Method

The indirect tensile splitting test used in the experimental

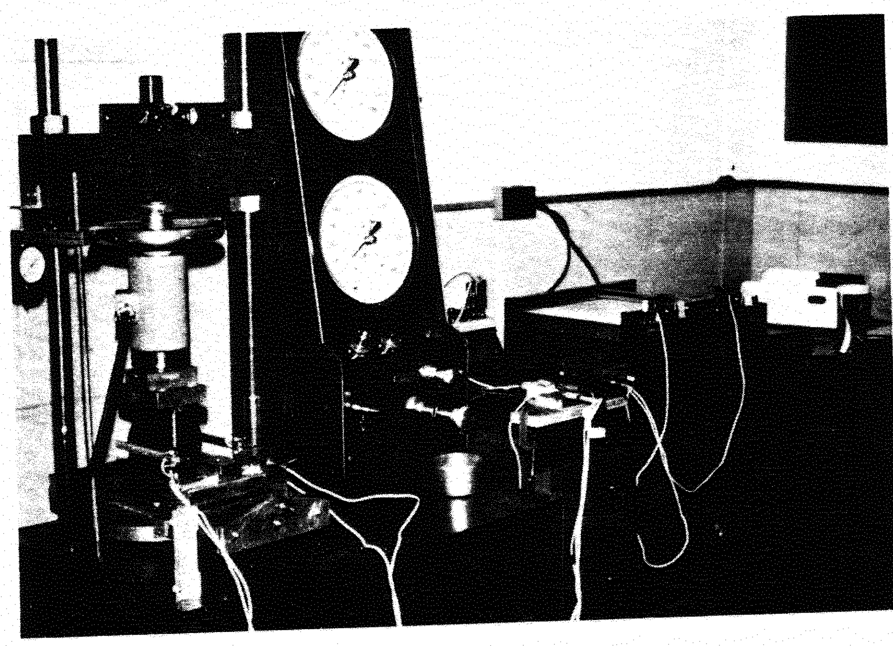


Figure 13. The Original Tensile Splitting Test Equipment

2. Upper and lower adaptor plates - special aluminum plates were attached to the universal testing machine to position the loading strips and the differential transducers.

3. Loading strips - a pair of curved-face loading strips made from aluminum were attached to the upper and lower adaptor plates.

4. Aluminum angles - a pair of aluminum angles were used to position and hold the differential transducers.

5. Load cell - a load cell, type U-1 Baldwin Lime Hamilton Corporation, of 10,000 lb (4535 kg) capacity was used to measure the applied load.

6. Deformation measuring device - a pair of Trans-Tek series 350, linear variable differential transducers were used to measure horizontal deformations.

7. Direct current power supplies - three Mirconta variable DC power supplies (Cat. No. 22-126) were used to power the load cell and the differential transducers.

8. Voltage indicator - a digital voltmeter, Data Precision Model 245, was used to set the various input and output voltage requirements.

9. Recorder - a Houston Instrument, Model Omnigraphic 2000, X - Y recorder plotter was used to plot the load-deformation curves.

10. Calibration device - a simple calibration device was used along with the digital voltmeter to calibrate the differential transducers.

Fig. 15 shows a detailed drawing of the tensile splitting test apparatus and Fig. 16 shows the electrical installation diagram for the apparatus. The transducer calibration device is shown in Fig. 17.

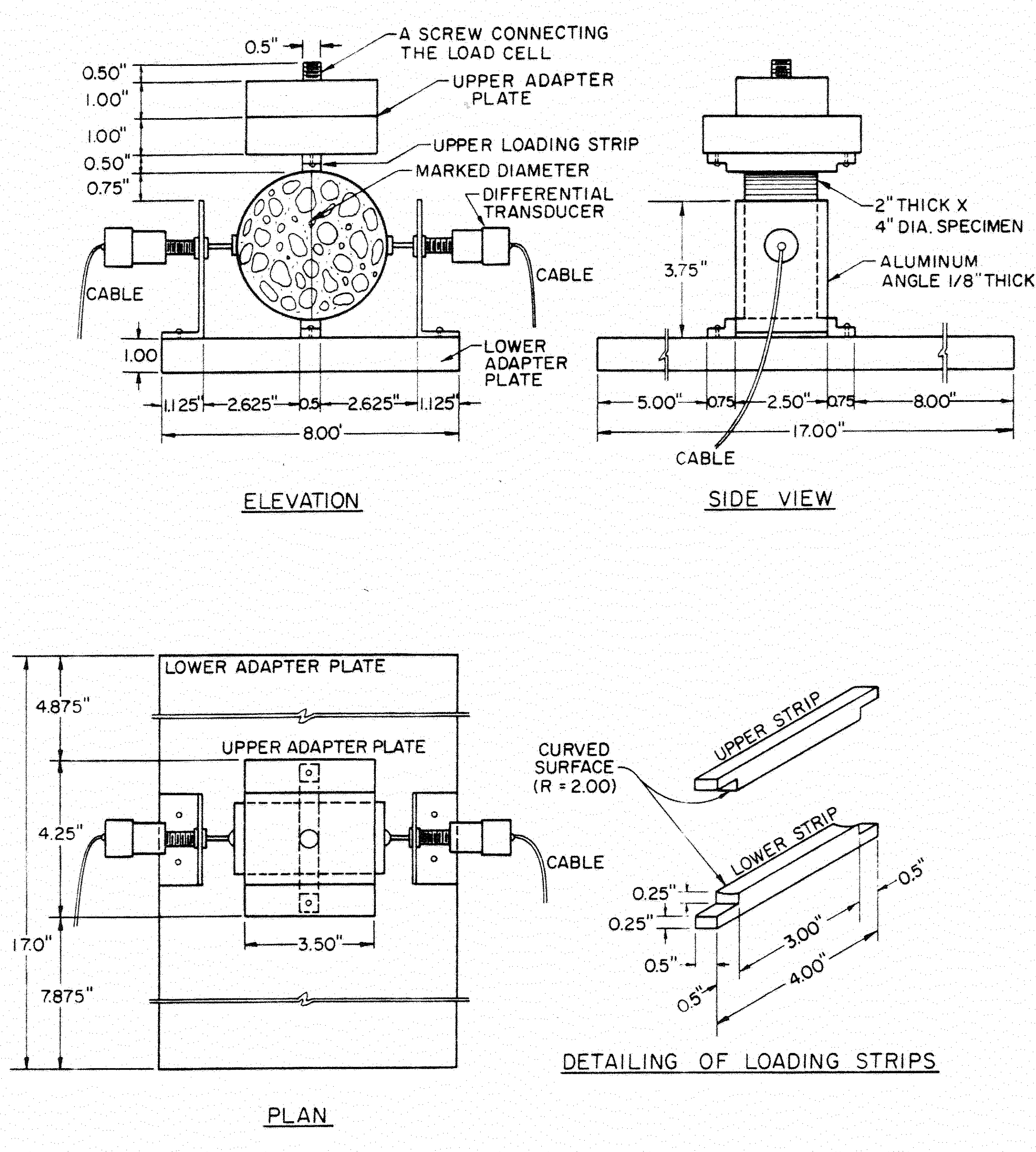


Figure 15. Detailed Drawing of the Tensile Splitting Test Apparatus

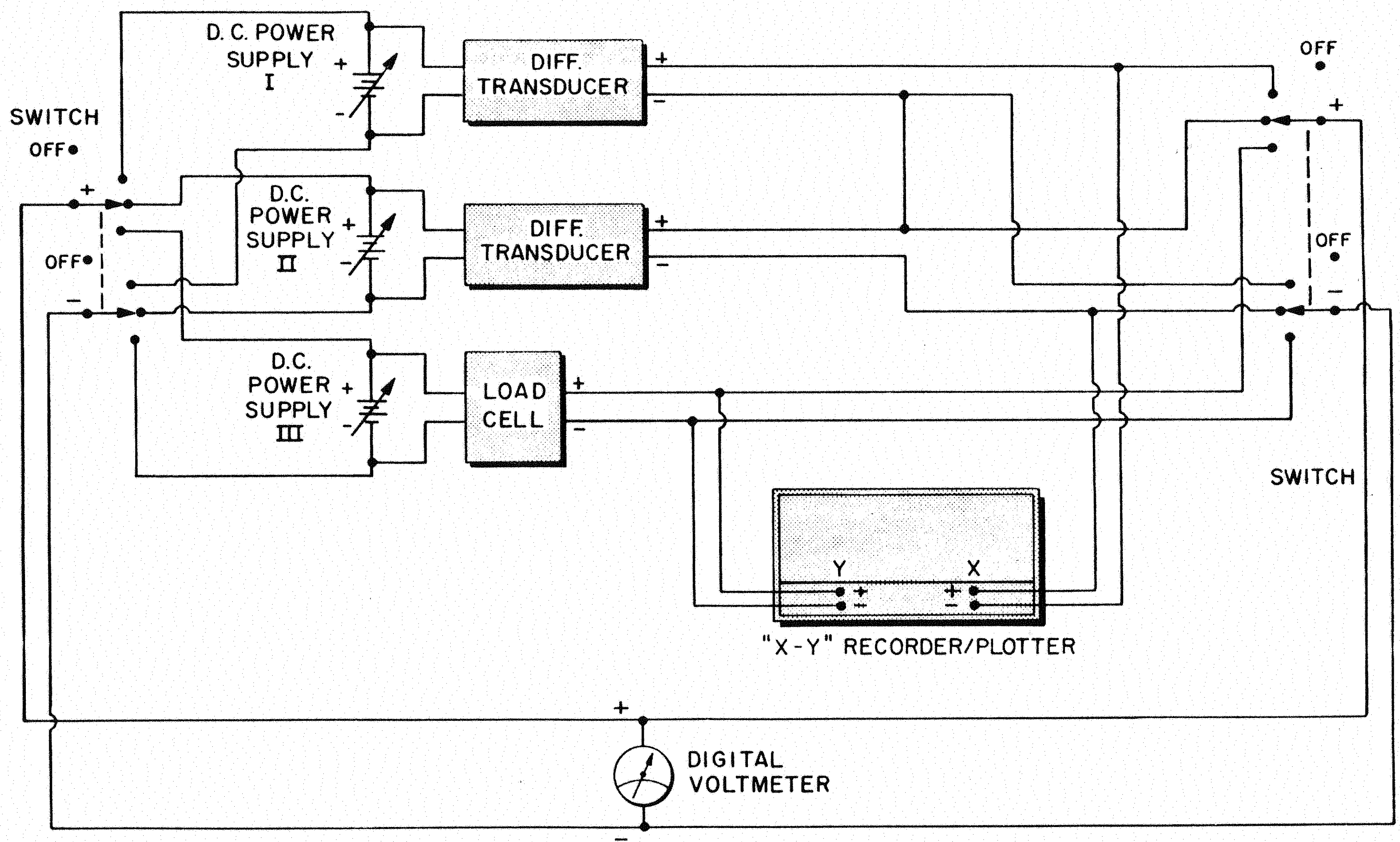


Figure 16. Electrical Installation Diagram

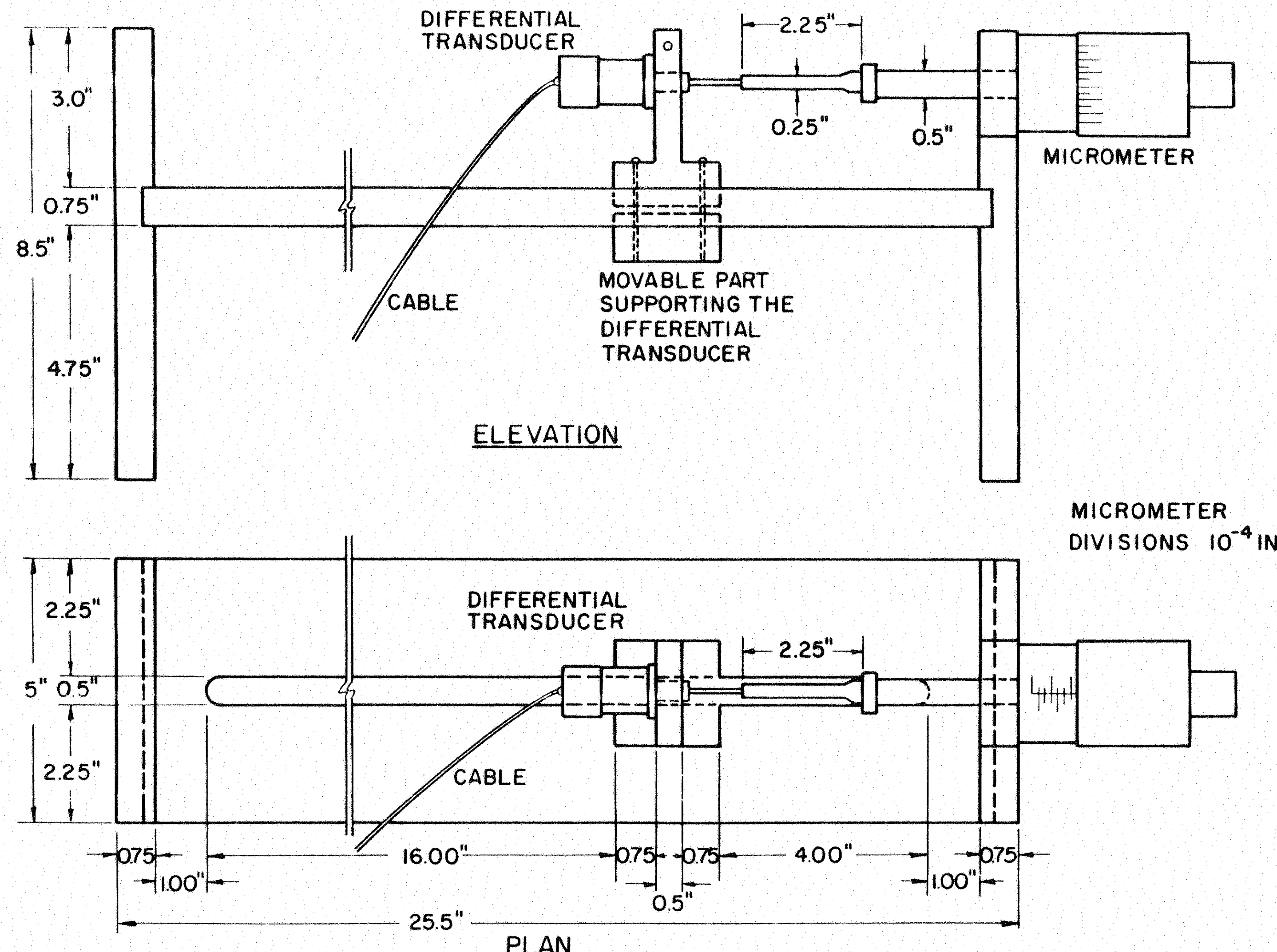


Figure 17. Calibration Device for the Differential Transducers

CHAPTER V

TEST PROCEDURES

The experimental program of this research was divided into two main phases relative to test procedures followed in surveying and core sampling at the field test sites and those employed in the laboratory to evaluate the desired properties of both field core samples and laboratory mixtures.

Field Test Procedures

Safety

The conduct of all field work on the State and Interstate highways was regulated to a great extent by the safety requirements for the research personnel. All field work was terminated whenever there was any form of bad weather conditions at the test site. No traffic control was required during the crack mapping and counting. However, research personnel wore high visibility safety vests and hats during the conduct of their work. Also, all the crack surveying work was carried out from the highway shoulders at the test sites. Special safety precautions were required during the coring operations at the test sites. One lane of the highway was kept open to traffic while the other was blocked for the core drill operations. At all the interstate test sites, appropriate warning signs were placed at least 880 yards (800 meters) in advance of the work area. This was followed by directional signals, flagmen and finally a physical barrier to close the lane being cored to traffic. The advanced warning signs were not required on the state highway test sites and the small traffic volumes were satisfactorily controlled by directional signals and flagmen. After completing the coring operations in the first lane, directional markers and barriers were switched to permit work in the other lane. All signing, signaling, blocking and flagging were carried out by personnel provided by the Research and other respective Divisions of the Oklahoma Department of Transportation.

Crack Survey

The crack survey included mapping and counting all the cracking patterns encountered in a 500 ft length chosen for study. Two research personnel working together were required to conduct this crack surveying. The first rolled a distance measuring device as he walked along one of the shoulders while the other carried a field data sheet similar to that shown in Fig. 18. Crack patterns were sketched on this sheet and classified according to type (multiple, full, half and part) as those previously indicated in Fig. 2. Each of these four types and the corresponding distance measured from the starting point of the 500 ft length were recorded in the appropriate columns of the field data sheet. At the end of each survey, the total number of multiple, full and half cracks were determined so that a cracking index for the particular site could be established. At every test site, some of the relatively new cracks were inspected and suitable ones were marked with yellow paint along the highway shoulder for further study. To show the severity and extent of some particular cracks, photographs were taken

School of Civil Engineering
Oklahoma State University

TRANSVERSE CRACKING IN FLEXIBLE PAVEMENTS

Field Data

Date:________ By:________ Highway Number:________ Highway Type:________

Survey Direction:________ Site Number:________ Site Description:________

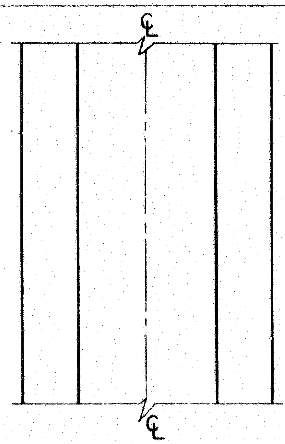
Dist., ft.	Cracking Data		General Rating of Surface Condition	Remarks
	Pattern	Class		
0				
10				
20				
30				
40				
50				
60				
70				
80				
90				
100				

Figure 18. Field Data Sheet for Crack Surveying and Mapping

and the corresponding cracks were identified in the field data sheet.

Core Drill Operations

All the core drill operations were scheduled in advance with the Research and Development Division of the Oklahoma Department of Transportation. For the preliminary crack study, a 6.0 in. (15.24 cm) diameter core drill was used to obtain core specimens at the end and/or middle of some particular part cracks at each test site. The angular speed and the rate of penetration of the core barrel were adjusted to minimize any disturbance of the pavement layers. To obtain laboratory test specimens, a 4.0 in. (10.16 cm) diameter core drill was used to cut 18 full-thickness cores at the selected wheelpath and non-wheelpath locations. At full cracks, these core locations were marked on the pavement surface about 8.0 in. (20.32 cm) away from the crack, as previously indicated in Fig. 5. After coring, each specimen was immediately wrapped in a plastic bag, appropriately identified and carefully stored for transporting to the laboratory.

Laboratory Test Procedures

Cutting Field Core Specimens

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 diamond edged concrete saw. The cut face of these specimens was trimmed to remove any surface irregularities. These specimens were then dried and stored prior to testing.

Bulk Specific Gravity

The bulk specific gravity of the laboratory test specimens was determined by weighing a specimen in air, weighing it in water, and computing its bulk specific gravity. Previous research conducted at Oklahoma State University (43) concluded that the bulk specific gravity values of field core samples were appreciably affected by water absorption. Based on this finding, the bulk specific gravity of field core specimens were determined with the samples coated with paraffin in compliance with ASTM method of test D-1188 (54). Tests were performed on at least three samples and an average value was computed.

Tensile Splitting Test

1) Thickness and Diameter Measurements: The thickness of each test specimen was measured at five points, at the center and at the ends of two arbitarily perpendicular diameters, and the average value was recorded as the specimen thickness. The diameter of the specimen was also determined as the average value of three measurements taken at approximately equal intervals along the specimen thickness. All the thickness and diameter measurements were made to the nearest 0.001 in. (0.0254 mm). In order to easily position the test specimen on the loading strips, chalk was used to mark a diametral loading line on one face of the specimen.

2) Cooling: A Lab-Line low-temperature cabinet or freezer was used to cool the test specimens to the desired temperature. A remote sensing temperature monitoring device (Scanning Tele-Therometer, YSI Model 47) was used to detect the temperature of the freezing cabinet,

the test room, and the test specimens. To monitor the temperature of the test specimens, four thermisters of the monitoring device were embedded at various points inside a dummy specimen similar in size and composition to the actual test specimens.

An approximate time of four minutes was required to take a test specimen from the freezing cabinet, position it, and run the test. A pilot study employing various low temperatures was performed to determine the rate of temperature increase at the different points of the dummy specimen. The results of this study indicated that an average rate of temperature increase of 1^{o} C/min could be assumed for the selected low temperature range of the test, i.e., 0^{o} to -10^{o} C. Therefore, the initial cooling temperature of specimens tested at this range was adjusted to four degrees less than the corresponding actual test temperature.

Test specimens were placed together with the dummy specimen in the freezing cabinet and the desired temperature was set on the cabinet controls. The temperature monitoring device was used to frequently check the temperature inside the cabinet and at the different sensor locations of the dummy specimen. The cabinet temperature was regulated, when necessary, and approximately four hours were required to achieve the desired testing temperature.

3) <u>Testing:</u> After making the necessary electrical connections on the load cell, the transducers, and the recorder, the power supplies were set to input the previously calibrated voltages of the load cell and the differential transducers. The appropriate horizontal and vertical scales of the X-Y recorder plotter were chosen and a 10.0 in. (25.4 cm) by 15.0 in. (38.1 cm) sheet of graph paper was attached to

the recorder.

The universal testing machine was warmed up for approximately 30 minutes before adjusting the deformation rate. By using a dial gage attached to the lower platen of the loading machine and a stop watch, the rate of deformation was regulated till a rate of 0.06 in./min (0.152 cm/min) was achieved.

After starting a stop watch, the specimen was removed from the freezer and carefully centered on the lower loading strip of the test apparatus. The upper loading head was slowly moved down until light contact was made with the test specimen. Position of the specimen was checked to make sure that the marked loading line was in a vertical plane passing through the center of the loading strips. The starting point of the load-deformation trace was marked by the recorder pen on the graph paper. When the elapsed time reached four minutes, load was applied to the test specimen until failure. The point at which the slope of the load-deformation trace reached zero was marked and the maximum load causing failure was recorded on the graph paper.

Rice's Specific Gravity

The maximum specific gravity of the test specimen mixtures was determined using the ASTM method of test D-2041 (54). This test procedure was slightly modified so available equipment in the O.S.U. Civil Engineering Asphalt Laboratory could be used. These modifications were listed in a previous research report (43).

Asphalt Extraction

Asphalt binders were extracted from pavement core samples in accordance with ASTM method of test D-2172 (54). Before processing, an asphalt content of six percent by total weight was assumed and the approximate weight of pavement material required to secure about 200 gm of recovered asphalt cement was calculated and used in the extraction. This quantity of asphalt was sufficient for four penetration samples of the asphalt binders at each test site. Test specimens were randomly chosen from the field cores so that representative asphalt cement samples would be obtained.

Asphalt Recovery

Asphalt was recovered from the extraction solution by a slightly modified Abson test method. Previous tests performed at the Asphalt Laboratory of the Oklahoma Department of Transportation using the ASTM method of test D-1856 (54) indicated that asphalt hardening occurred during the course of the recovery procedure. This asphalt hardening was considered to be due primarily to the combined effect of high temperature and the time during which this temperature was maintained (160^{o} C and 15 minutes, respectively). To investigate this effect, trials were made employing three different penetration grades of asphalt cement (40-50, 85-100 and 120-150). The original penetration of each asphalt cement was determined in accordance with ASTM method of test D-5 (54). Each asphalt cement was mixed with a sufficient quantity of a solvent (1,1, 1-trichloroethane) so a solution similar to that obtained from the extraction test could be made. Abson recovery tests were performed on the prepared solutions and all possible combinations of three temperature levels (150, 155 and 160^{o} C) and three time periods (5, 10 and 15 minutes) were tried. After each test, the penetration of the recovered asphalt was determined and compared with that of the original asphalt.

The results of this investigation substantiated the previous findings of ODOT research personnel. The amount of hardening was found to be a function of both the test temperature and time. Also, the observed amount of hardening of the relatively soft asphalt grades was greater than that of the harder asphalts. This finding can be attributed to the higher content of resins and oils of the soft asphalt cement as compared to those of the harder grades. For the expected penetration values of field asphalts, linear interpolation of the test results indicated that hardening of the field asphalts during the test procedure could be either eliminated or minimized by reducing the temperature to about 155^{o} C and the time to 6 minutes.

An International (Size 2, Model V) centrifuge was used to process the extraction solutions prior to the asphalt recovery procedure. The angular speed of this centrifuge was regulated to produce the centrifugal force specified in the ASTM standard test.

Recovered Asphalt Properties

Three asphalt cement samples recovered from each test site were evaluated in terms of their rheological properties. The following tests on these samples were performed at the Oklahoma Department of Transportation Asphalt Laboratory:

1. Penetration test, ASTM D-5 (54)

2. Kinematic viscosity test, ASTM D-2170 (54)

3. Absolute viscosity test, ASTM D-2171 (54)

The softening point of the asphalt samples was determined at the O.S.U. Civil Engineering Asphalt Laboratory by the ring-and-ball procedure conforming to ASTM method of test D-36 (54).

CHAPTER VI

TEST RESULTS AND DISCUSSION

Field Core Samples

Tensile Properties at Low Temperatures

The tensile splitting test results were used as input data for a statistical analysis of variance. Tests for evidence of real differences in the observed values and an estimation of the magnitude of such differences were conducted using the Statistical Analysis System (SAS) computer program (16). The results of these tests indicated the Observed Significance Level and acceptance or rejection of the null-hypothesis (no differences) was based on a reasonalbe significance level. For this study, a value of 0.05 was considered as the significance criterion for the rejection or acceptance of the hypothesis of no differences. The results of this analysis and of the correlation studies with the observed degrees of cracking (Appendix A) are discussed for the three tensile properties investigated.

Tensile Strength: The average tensile strength values are given in Table I and illustrated in Fig. 19. Analysis of variance of all test results indicated a strong evidence of location (wheelpath and non-wheelpath) differences in the tensile strength values and the observed significance level was 0.0014. Fig. 19 shows that the tensile strength

TABLE I

AVERAGE TENSILE STRENGTH OF FIELD CORE SPECIMENS

Site No.	Average Tensile Strength (σ_{TF}), psi					
	Wheelpath			Non-Wheelpath		
	-10°C	-5°C	0°C	-10°C	-5°C	0°C
1	346.493	262.400	247.659	336.119	239.109	223.114
2	307.558	266.903	233.484	252.685	247.629	215.526
3	344.593	339.630	217.713	340.220	308.527	227.333
4	266.994	252.431	252.263	266.432	235.613	218.533
5	366.904	347.032	323.727	385.208	328.192	299.024
6	393.775	338.680	290.535	418.583	329.180	251.143
7	406.310	286.475	248.348	345.847	277.801	272.088
8	412.611	353.420	305.426	366.612	357.580	304.768
9	355.571	332.091	230.342	384.394	259.606	266.930

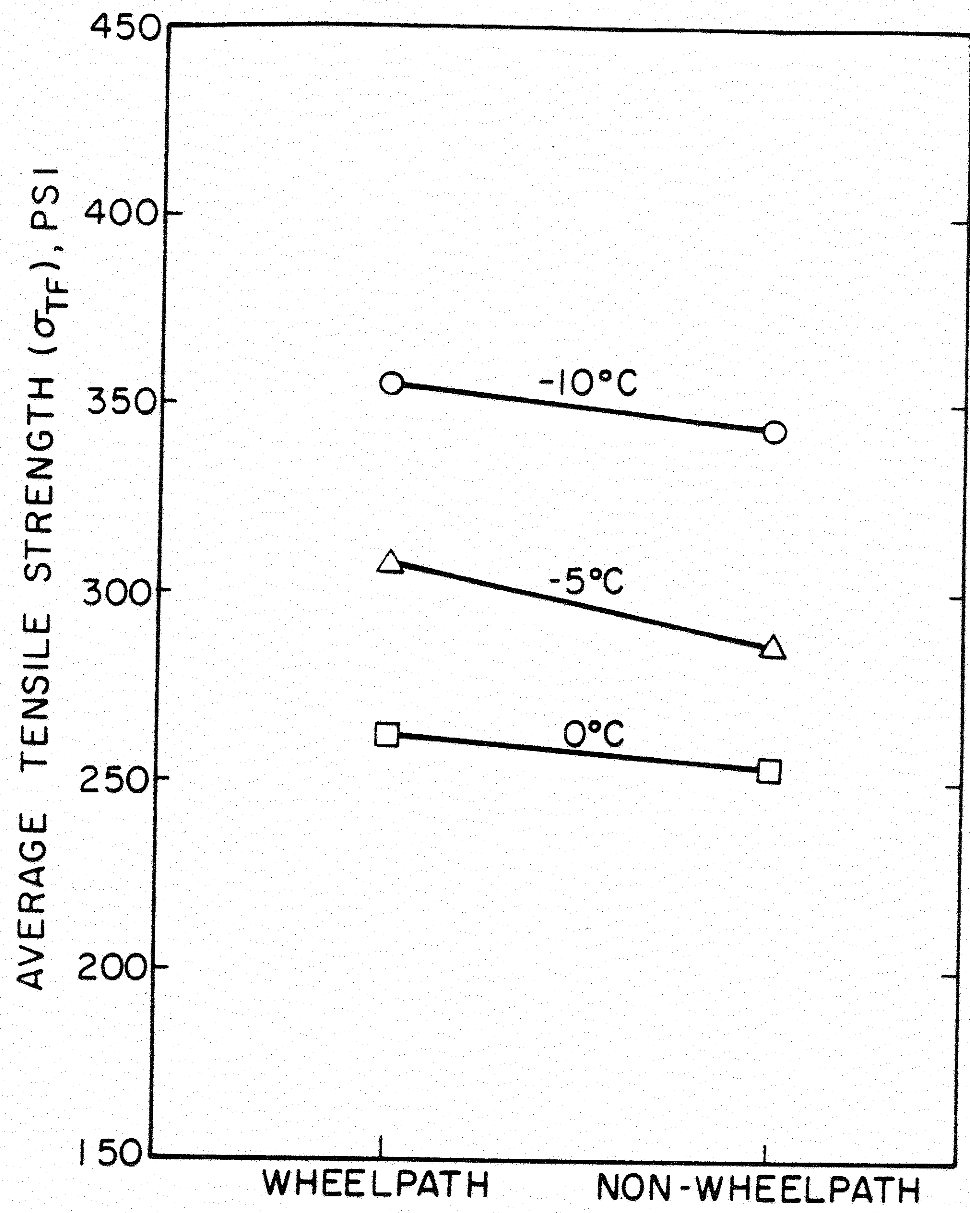


Figure 19. Average Tensile Strength of Wheelpath and Non-Wheelpath Specimens at Various Low Temperatures

of wheelpath specimens were considerably greater than those of the non-wheelpath specimens. The high tensile strengths associated with wheelpath specimens can be attributed to the relatively higher pavement densities developed under traffic wheel loads.

Test temperature had a very significant effect on the tensile strength values. Average tensile strengths at -10°C were noticeably higher than those at -5° and 0°C, respectively. The observed significance level was 0.0001. These results 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. Consequently, an increase in the average tensile stress at failure could be expected.

The analysis of variance showed that the greated variation in the tensile strength values was attributed to differences in properties of the test sites (blocks) and the observed significance level was 0.0001. In general, higher tensile strengths were observed for the interstate sites (Sites 5 to 9). This may indicate the importance of the quality and adequancy of pavement design and construction procedures on the service behavior at low temperatures.

A correlation study was made to investigate the general trend of the tensile strength-cracking index relationship. To minimize the effect of variation in material properties, this correlation study was performed for wheelpath and non-wheelpath specimens separately. Data was fed into a Hewlett-Packard Calculator Plotter (Model 9862 A) 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. The regression lines, correlation coefficients (r) and corresponding observed significance levels ($\hat{\alpha}$) are illustrated in Figs. 20 and 21 for wheelpath and non-wheelpath specimens, respectively.

In general, test sites with a high degree of cracking showed lower tensile strength values. However, a considerable scattering of tensile strength values was observed. A significant part of the scatter involved is probably due to the natural non-homogenity of the materials which, in turn, reduced the observed correlation coefficients.

The results of this analysis indicate that pavement surface mixtures with high tensile strengths will generally be more resistant to fracture at low temperatures. However, tensile strength should not be the only variable considered since the pavement can crack due to excessive tensile strains.

Tensile Strain at Failure: Table II and Fig. 22 show the average values of tensile strain at failure for wheelpath and non-wheelpath specimens. As can be seen in Fig. 22, tensile strains of wheelpath specimens were considerably higher than those of the non-wheelpath specimens. Analysis of variance of test results showed that the observed significance level associated with location differences was 0.0042. With the exception of the average tensile strain value of wheelpath specimens at -10°C, average tensile strain at failure significantly decreased as temperature decreased. The observed significance level was 0.034. This substantiated the effect of service temperature on the behavior of the asphalt concrete mixtures. As discussed earlier, lower tensile failure strains at relatively low temperatures can be related to the elastic rather than the viscoelastic response of the asphalt mixture. No significant interaction was found between temperature and location ($\hat{\alpha}$ = 0.6).

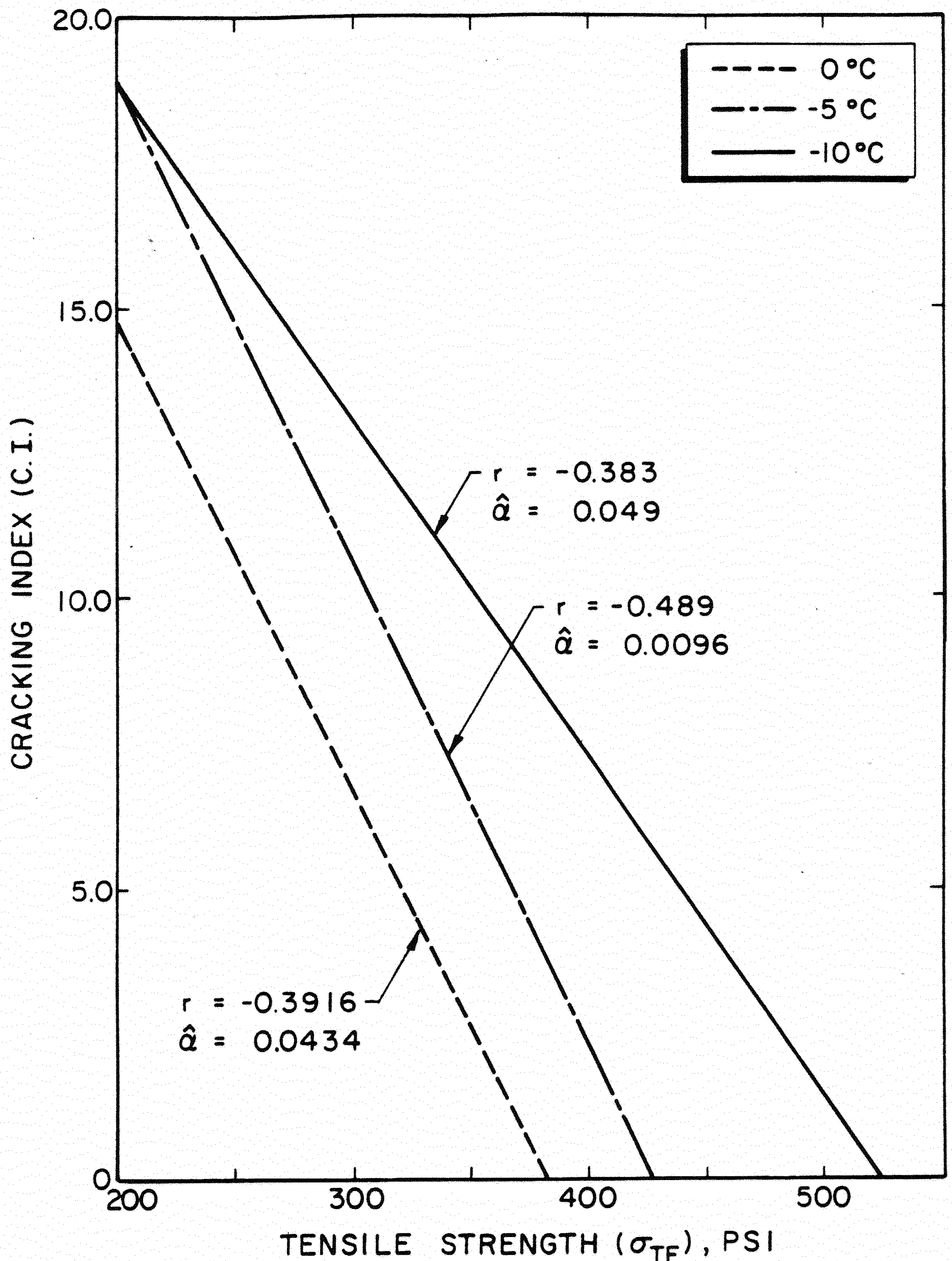


Figure 20. Relationship Between Tensile Strength of Wheelpath Specimens and Cracking Index

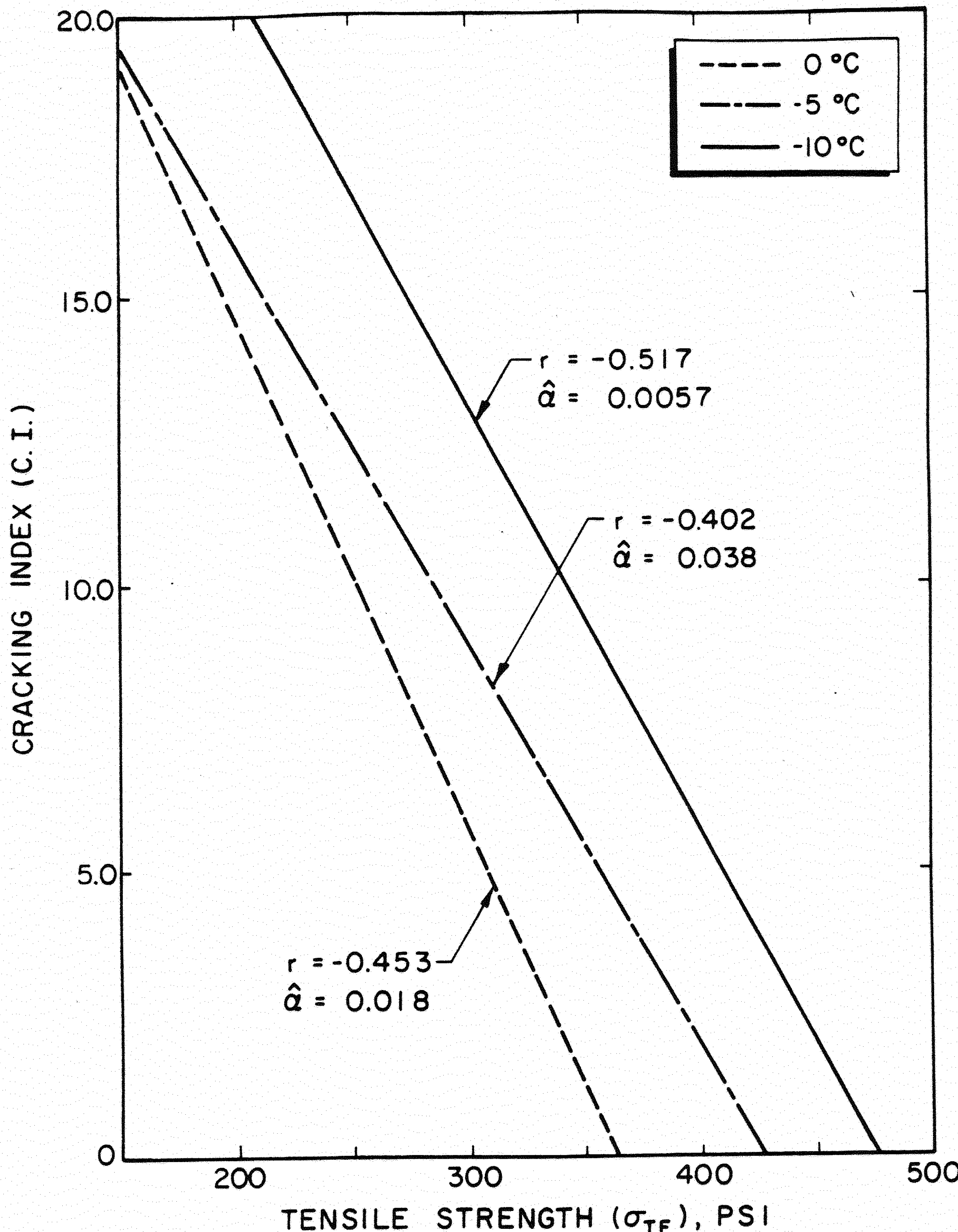


Figure 21. Relationship Between Tensile Strength of Non-Wheelpath Specimens and Cracking Index

TABLE II

AVERAGE TENSILE STRAIN AT FAILURE OF FIELD CORE SPECIMENS

Site No.	Average Tensile Strain At Failure (ε_{TF}), 10^{-3} in./in.					
	Wheelpath			Non-Wheelpath		
	-10° C	-5° C	0°C	-10° C	-5° C	0°C
1	3.014	0.934	2.671	2.287	1.674	1.718
2	2.573	1.357	2.219	1.897	1.445	1.646
3	3.215	2.109	2.236	1.765	1.561	2.066
4	0.934	0.969	1.474	0.467	0.872	0.744
5	3.947	4.934	5.542	3.711	5.101	5.348
6	0.930	1.683	2.288	0.564	1.740	1.758
7	1.683	2.961	2.707	1.903	2.454	2.594
8	5.498	4.965	6.393	4.113	3.933	6.617
9	4.228	4.780	3.558	3.947	3.871	4.728

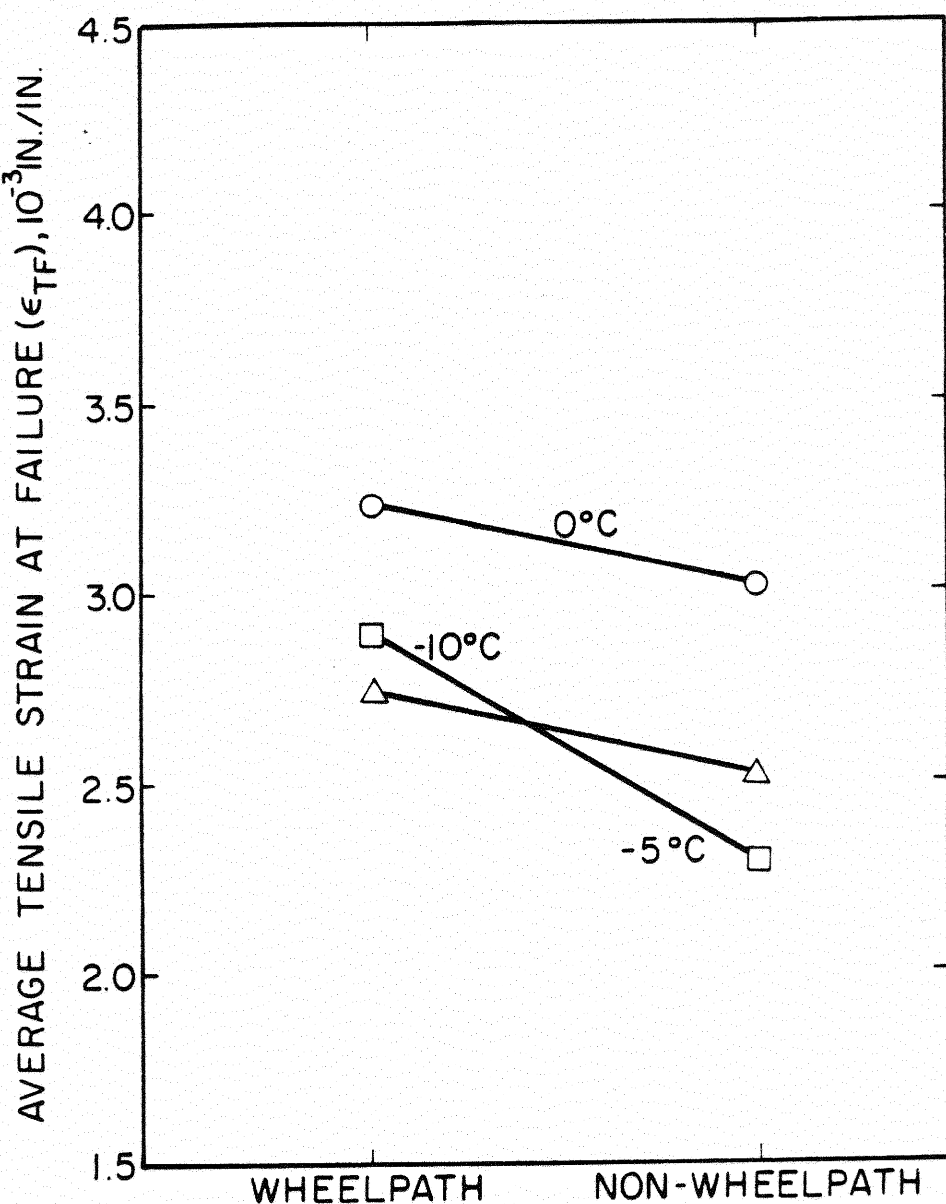


Figure 22. Average Tensile Strain at Failure of Wheelpath and Non-Wheelpath Specimens at Various Low Temperatures

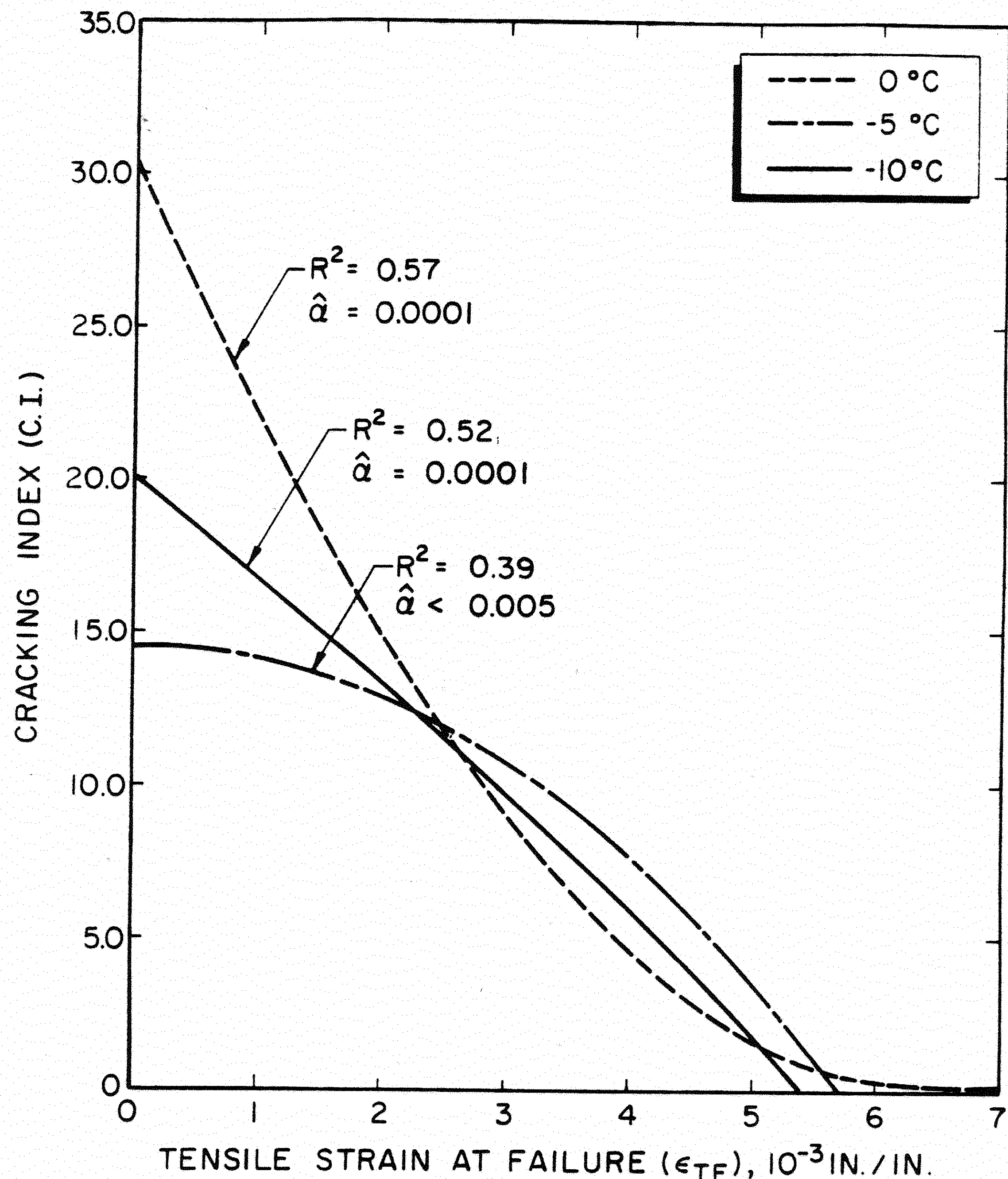


Figure 23. Relationship Between Tensile Failure Strain of Wheelpath Specimens and Cracking Index

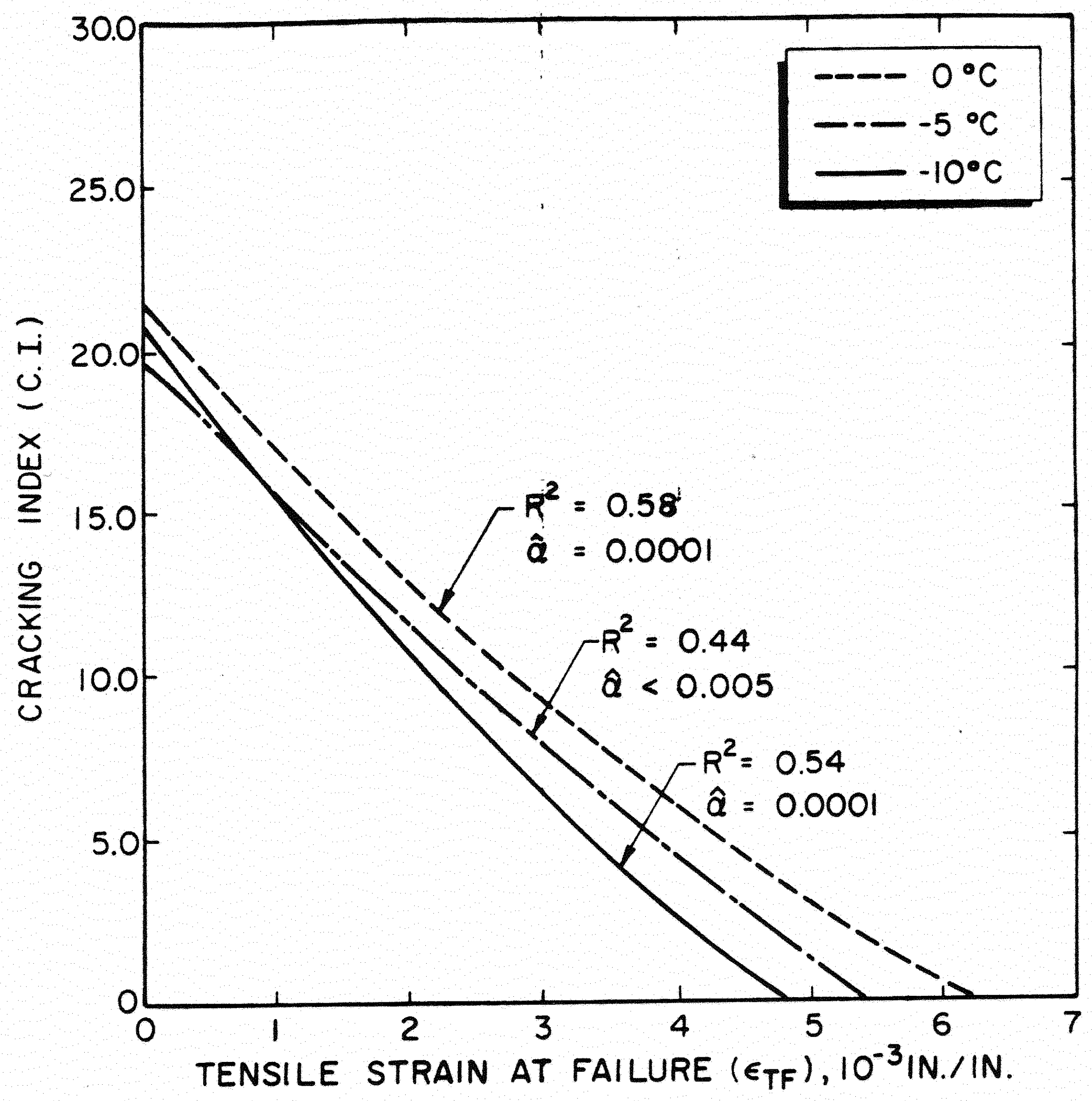


Figure 24. Relationship Between Tensile Failure Strain of Non-Wheelpath Specimens and Cracking Index

TABLE III

AVERAGE ULTIMATE FAILURE STIFFNESS OF FIELD CORE SPECIMENS

Site No.	Average Ultimate Failure Stiffness (S_{TF}), 10^3 psi					
	Wheelpath			Non-Wheelpath		
	-10°C	-5°C	0°C	-10°C	-5°C	0°C
1	117.434	324.441	94.25	149.479	152.384	131.879
2	123.057	212.991	111.721	141.142	190.511	140.622
3	107.408	193.038	100.947	208.728	231.690	115.209
4	229.236	322.280	193.252	322.280	487.935	294.129
5	81.242	70.672	59.150	70.672	64.651	55.981
6	474.945	327.221	131.938	745.495	276.461	142.975
7	305.623	100.072	96.364	251.124	120.022	105.582
8	76.064	74.925	48.127	94.551	92.319	46.135
9	85.988	69.811	64.657	98.485	68.459	56.553

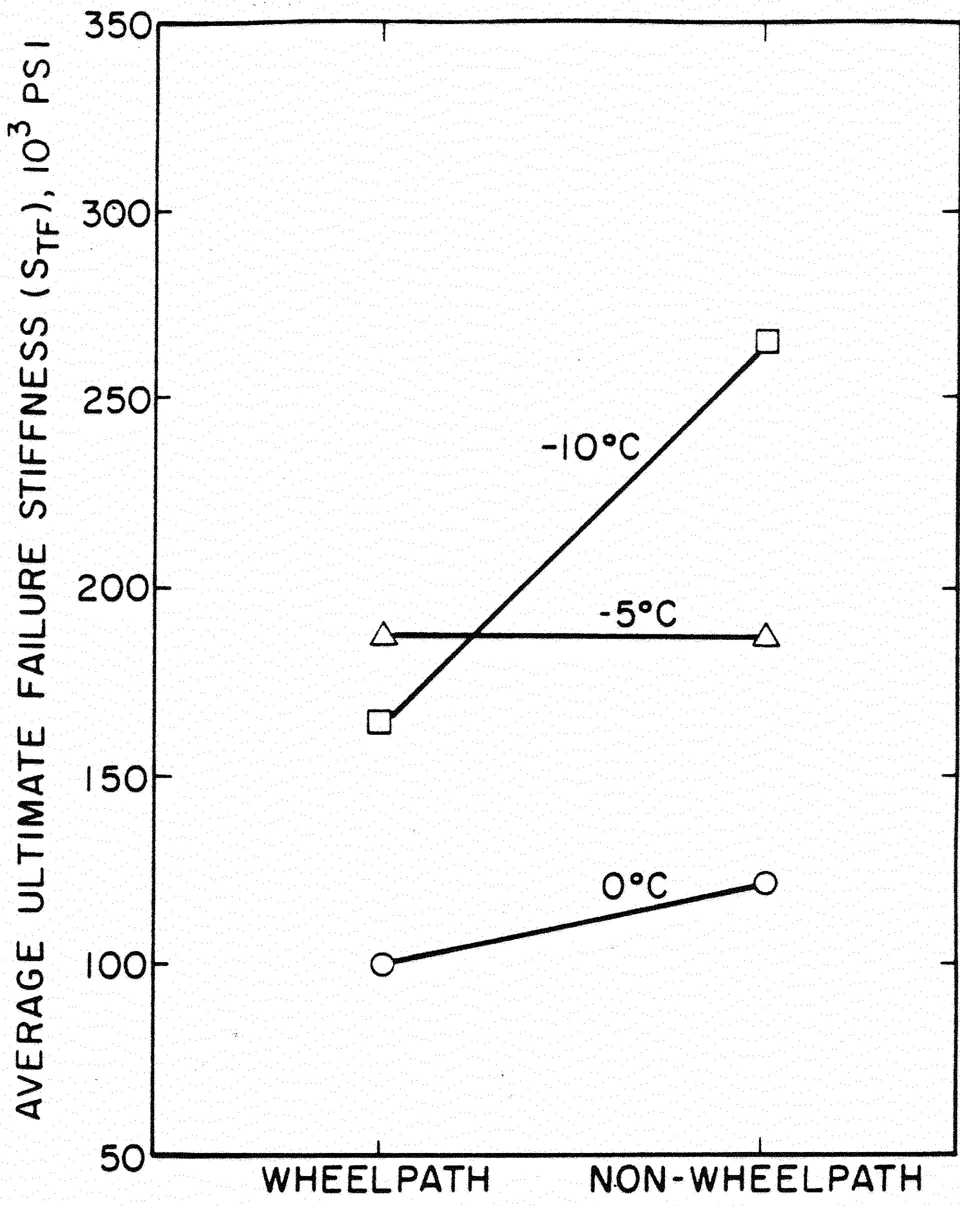


Figure 25. Average Ultimate Failure Stiffness of Wheelpath and Non-Wheelpath Specimens at Various Low Temperatures

sites were significantly different with respect to their ultimate failure stiffness responses.

Results of the correlation analysis are shown in Fig. 26. The degree of cracking or cracking index was proportional to the ultimate failure stiffness at all test temperatures and test sites with a high degree of cracking generally exhibited the higher failure stiffness values. However, it should be noted that the determined correlation coefficients were relatively small. The best correlation between test results and degree of cracking was that of 0°C.

Stiffness Moduli of Recovered Asphalt Cements and Asphalt Mixtures

McLeod's method (23) was employed to calculate the stiffness moduli of recovered asphalt cements. At first, it was felt that a minimum service temperature of -10°C could be assumed for the stiffness calculations. This temperature was chosen in order to study the relationship between the calculated stiffness values and the tensile properties of the mixtures at the same temperature. However, a study of the climatological data (55) over a 73-year period revealed that the lowest minimum air temperature recorded in Oklahoma was -17°F (-27.22 C°). Based on temperature data reported in another research study (27), the temperature at a pavement depth of 2 in. was about 7° to 8°F higher than the air temperature. The temperature at a pavement depth of 2 in. was specified by McLeod to ensure that a substantial thickness of the asphalt structure was being subjected to the contraction stresses and strains developed at the critical low temperature. Consequently, modulus of stiffness values were calculated at a temperature of -10°F.

The average stiffness modulus and the rheological properties of the

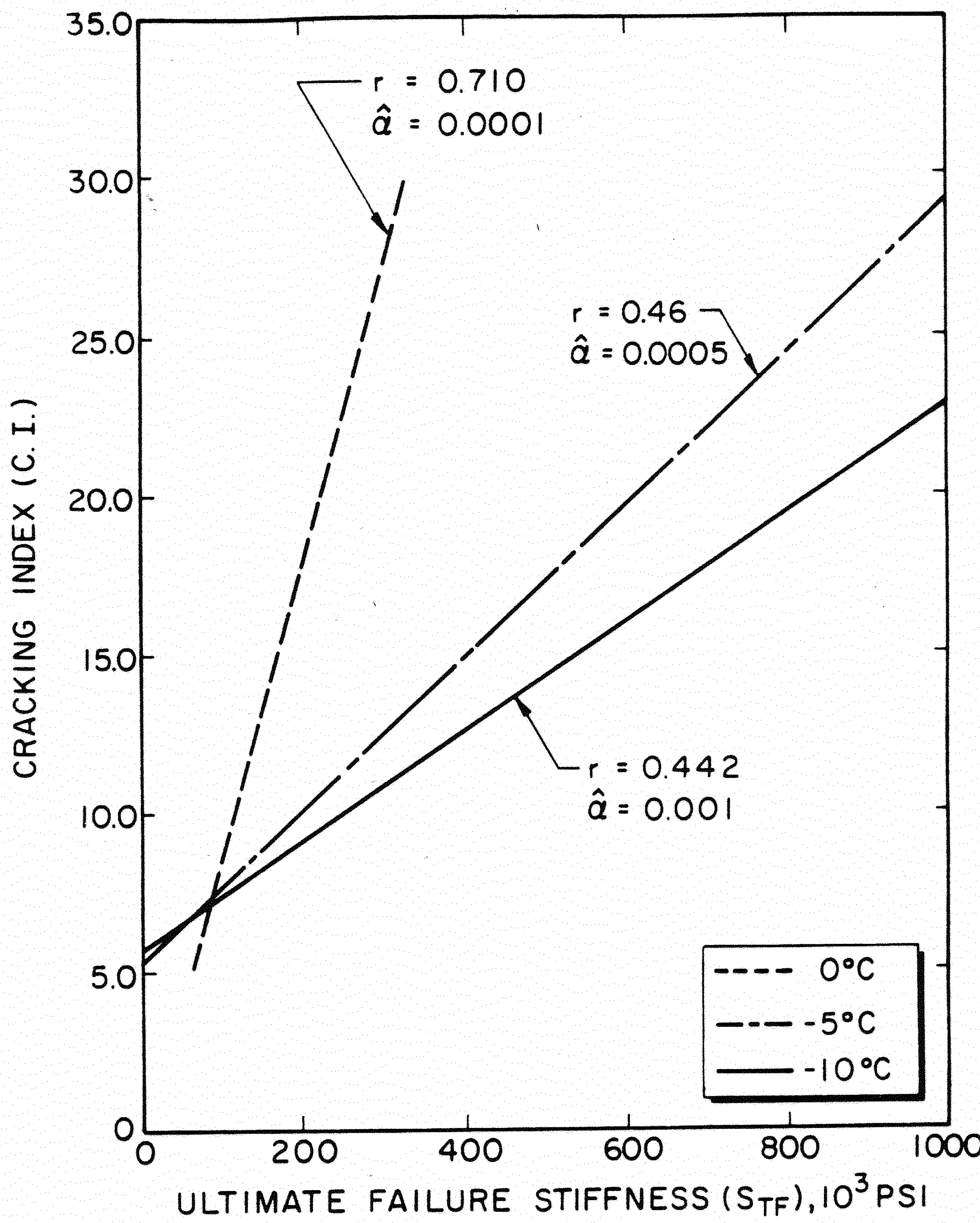


Figure 26. Relationship Between Ultimate Failure Stiffness and Cracking Index

recovered asphalt cements are given in Table IV. Also, the average stiffness moduli of field asphalt mixtures are shown in Table V along with the composition data of individual test specimens from the respective sites.

Correlation studies indicated that the low-temperature stiffness moduli of the recovered asphalt cements could be correlated with the observed degree of cracking (Fig. 27). Pavement sections with a high degree of cracking were, generally, those having the stiffer asphalt cements in the surface layer. Only Site 7 showed an exception to this trend. The cracking index of this section was 20.0, but the stiffness modulus was relatively low (2916 psi). It is possible that the high cracking frequency of this site was associated with a subgrade problem or other related factors. If the data of this site is disregarded, the coefficient of determination goes up to 0.59. This is a remarkably high value considering the variables that underlie this generalized relationship.

A similar relationship was found for the stiffness moduli of the field mixtures (Fig. 28). However, the coefficients of determination associated with this relationship were relatively smaller than those of the previous one. This can be attributed to the variation in the other mix properties of the surfacing at the individual test sites, i.e., variation in asphalt content, specific gravity and percent density. Again, if the stiffness moduli of the Site 7 mixtures are disregarded, the coefficients of determination to 0.16 and 0.18 for the wheelpath and non-wheelpath stiffness moduli versus cracking index relationships, respectively.

TABLE IV

RHEOLOGICAL PROPERTIES AND STIFFNESS MODULI OF RECOVERED ASPHALT CEMENTS

Site No.	Ring & Ball Softening Point, F°	Penetration at 77°F	Absolute Viscosity at 140°F (Poise)	Kinematic Viscosity at 275°F (C. St.)	Stiffness Modulus at -10°F, psi
1	167.72	26.00	651,700.0+	5,408.67	3738
2	160.52	32.00	65,327.0	3,063.00	3558
3	136.04	44.00	22,311.0	1,214.33	4876
4	173.04	18.00	218,360.0	2,353.33	14226
5	124.52	57.00	3,606.0	546.33	2703
6	139.55	41.50	20,630.0	1,146.50	3665
7	128.57	58.50	3,853.0	536.50	2916
8	126.32	59.33	4,058.3	639.00	2416
9	123.26	62.00	2,610.7	502.00	2688

TABLE V

COMPOSITION AND STIFFNESS MODULI OF FIELD SPECIMENS

Site No.	Asphalt Content, %	Bulk Sp. Gravity		Per Cent Density		Volume Conc. of Agg., ($\bar{C}_v$)		Stiffness Mod., psi	
		WP*	NWP**	WP	NWP	WP	NWP	WP	NWP
1	5.619	2.368	2.327	94.845	93.992	0.8416	0.8358	451,470	415,086
2	5.920	2.325	2.327	93.803	93.211	0.8268	0.8212	329,766	306,309
3	5.605	2.414	2.380	97.236	96.989	0.8604	0.8595	703,958	696,339
4	5.884	2.177	2.172	91.854	90.871	0.8185	0.8097	737,773	664,394
5	5.475	2.471	2.405	98.827	96.178	0.8631	0.8856	516,405	453,347
6	4.867	2.412	2.396	97.031	96.047	0.8745	0.8703	793,486	721,770
7	5.834	2.411	2.376	98.368	96.467	0.8569	0.8516	480,532	436,460
8	5.508	2.466	2.420	97.873	96.630	0.8612	0.8567	463,585	425,840
9	5.573	2.420	2.393	98.257	96.710	0.8627	0.8562	510,891	454,556

* Wheelpath Specimens

** Non-Wheelpath Specimens

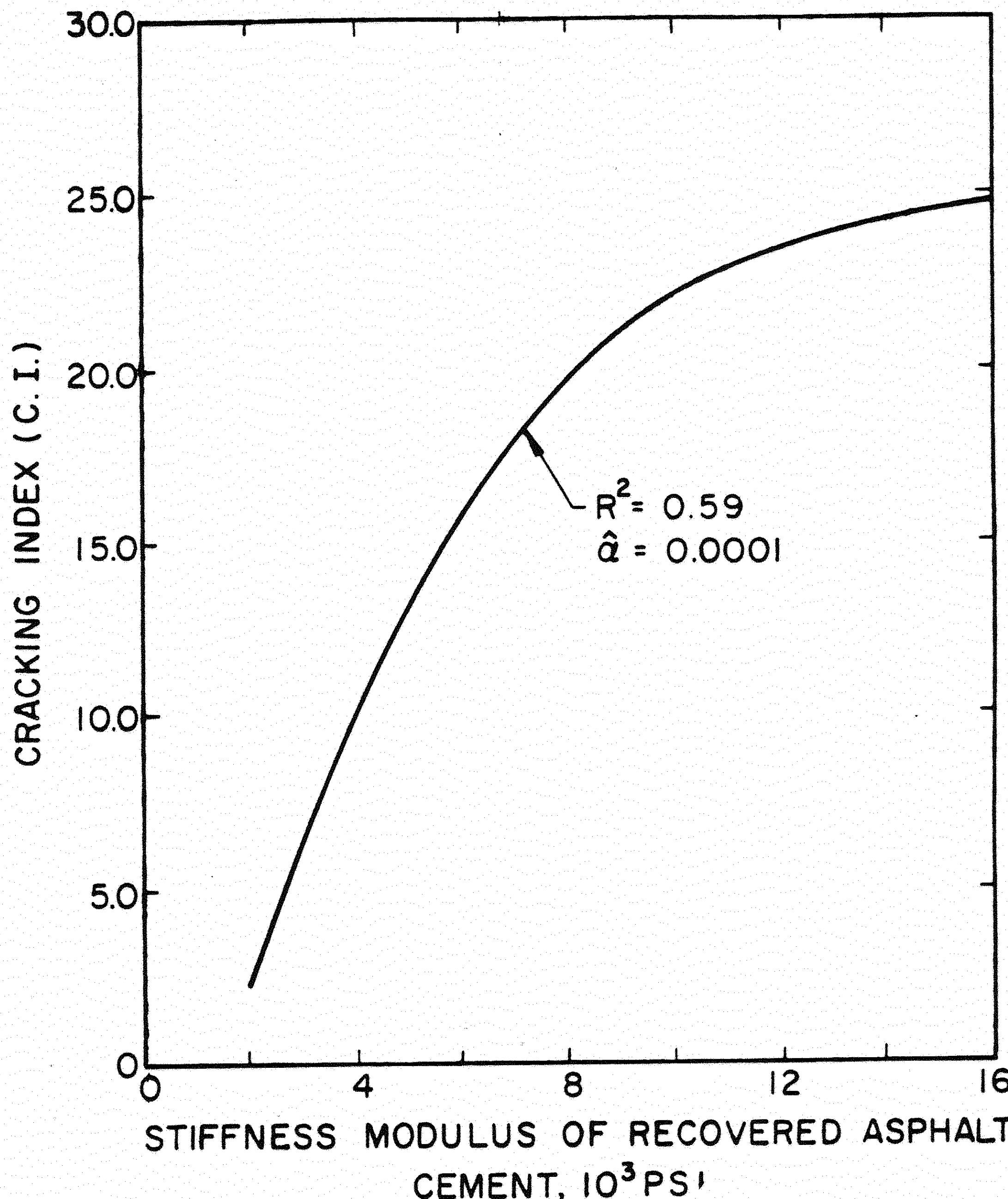


Figure 27. Relationship Between Stiffness Moduli of Recovered Asphalt Cements and Cracking Index

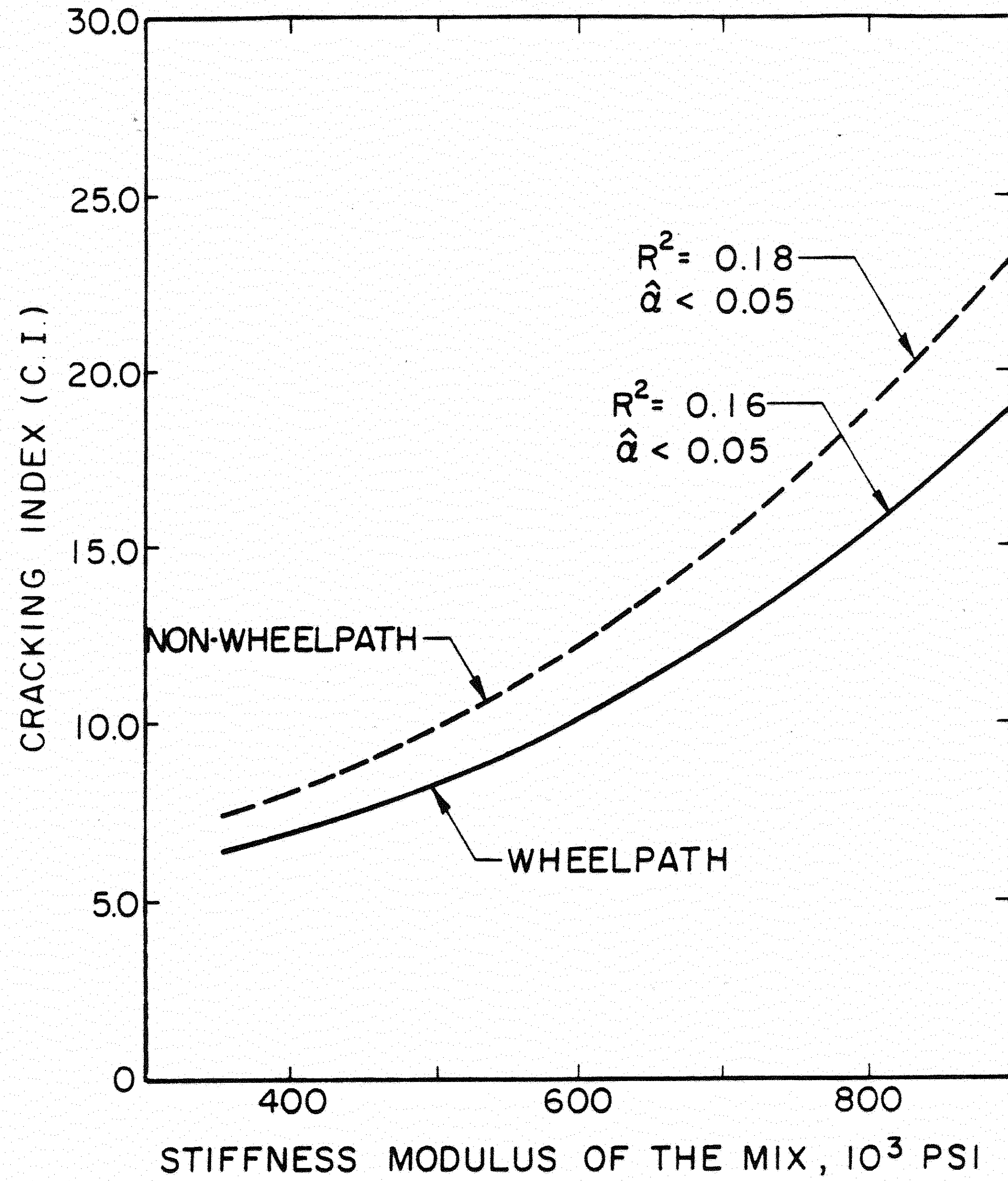


Figure 28. Relationship Between Stiffness Moduli of Field Mixtures and the Cracking Index

Laboratory Mixtures

Tensile Properties at Various Temperatures

A factorial experiment was designed to investigate the effect of asphalt grade and asphalt content on the tensile properties of fresh asphalt concrete mixtures at different temperature levels. Results of the tensile splitting tests were analysed by the corresponding (SAS) computer program. The findings of this analysis are discussed in the following paragraphs under the three tensile properties studied.

Tensile Strength: The average tensile strength values are given in Table VI. Also, Fig. 29 illustrates the effect of temperature, asphalt grade and asphalt content on the average tensile strength values. Based on these data, it seems that the temperature had a remarkable effect on the tensile strength or the stresses developed at failure. These stresses markedly increased as temperature decreased. Analysis of variance substantiated this finding and the observed significance level of the main temperature effect was 0.0001 indicating a strong evidence of temperature differences in the tensile strength values.

At each temperature level, higher average tensile strengths were assocaited with the stiffer asphalt cements. For instance, mixtures prepared with the 91-penetration asphalt cement showed the highest tensile strength values at all temperatures. Also, tensile strengths of the 124-penetration mixes were higher than those of the 160-penetration mixes at all temperatures. The observed significance level assocaited with the asphalt grade effect was 0.0001. However, it should be noted that the rate of increase in the tensile strength was dependent on the temperature, e.g., the increase in tensile strength values of the 91-

TABLE VI

AVERAGE TENSILE STRENGTH OF LABORATORY SPECIMENS (psi)

Asphalt Pen. Grade	91 pen.			124 pen.			160 pen.		
Asphalt Content, %	4.5	5.0	5.5	4.5	5.0	5.5	4.5	5.0	5.5
a. Temp. = +20°C	58.433	71.833	64.433	49.467	60.867	46.933	24.139	28.889	29.001
b. Temp. = 0°C	242.467	239.190	235.587	218.783	215.146	186.693	187.054	153.673	132.409
c. Temp. = -10°C	381.777	390.913	388.434	357.314	344.123	291.794	320.294	319.424	289.716

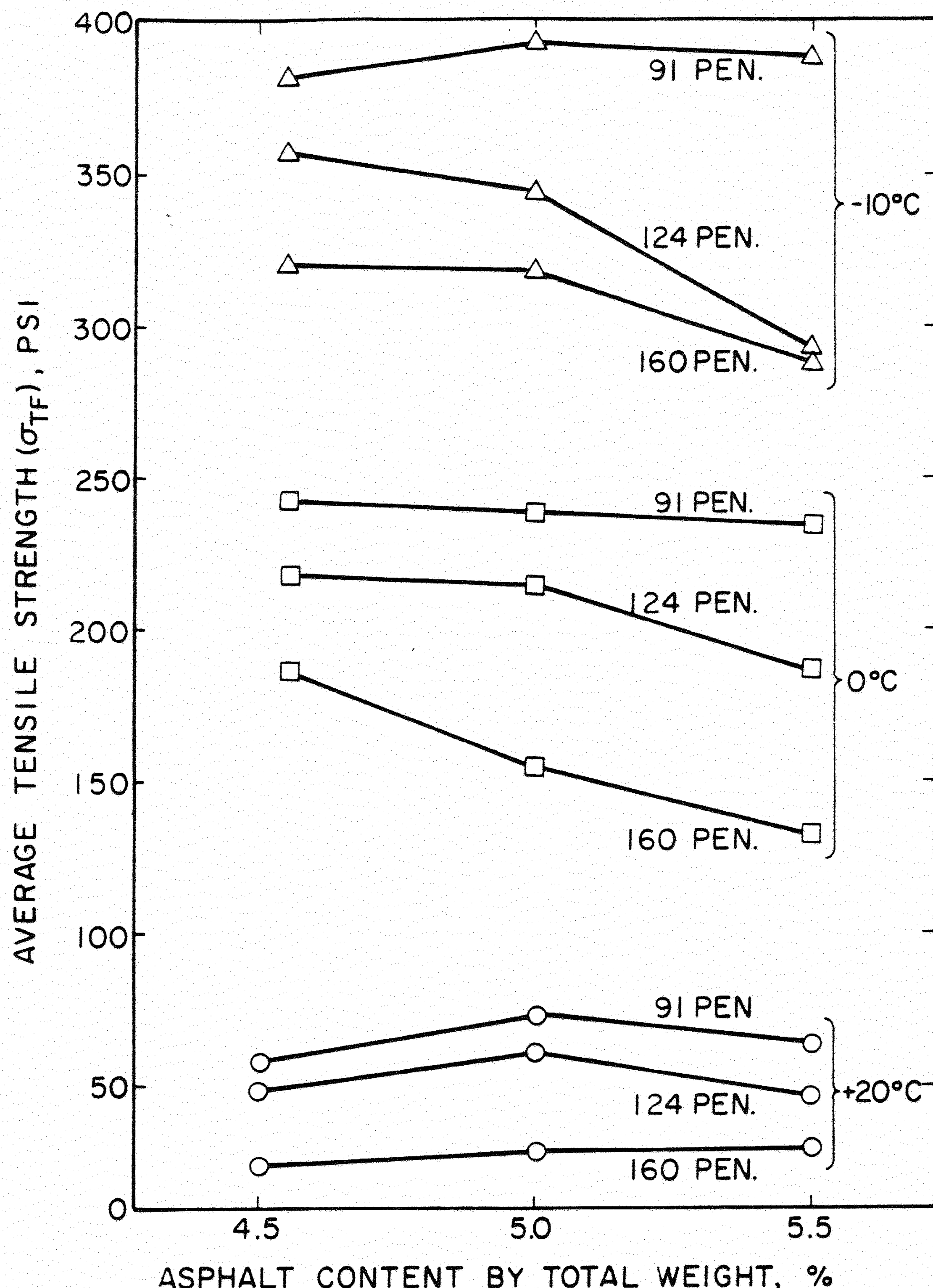


Figure 29. Average Tensile Strength Versus the Asphalt Content of the Mix

penetration asphalt mixtures was noticeably higher at -10°C than that at 0° and 20°C. The analysis of variance confirmed this finding and indicated a significant interaction between the temperature and the asphalt grade factors. The observed significance level of this interaction was less than 0.005.

Asphalt content had a significant effect on the observed tensile strengths. The observed significance level was less than 0.005. However, this effect was apparently dependent on the levels of the other two factors as may be indicated in Fig. 29. For instance, the maximum tensile strength values at 20°C of the 91 and 124-penetration mixture groups were achieved at 5.0 percent asphalt content, while those of the 160-penetration asphalt mixtures were attained at 5.5 percent asphalt content. With the exception of the 91-penetration asphalt mixtures at -10°C, average tensile strength values generally decreased as asphalt content increased. The amount of decrease was considerably greater at 5.5 percent asphalt content for the 124 and 160-penetration mixes than that of the 91-penetration mix. It appears that, as asphalt content increases, stiffness of the mixture decreases and relatively lower tensile stresses can be developed.

Tensile strength results generally showed that higher tensile strengths (stresses at failure) were associated with stiffer (lower penetration or higher viscosity asphalt cements. As previoudly indicated, mixtures with high tensile strengths may be more resistant to deformation and cracking if they exhibit similarly high strains at failure.

Tensile Strain at Failure: Table VII shows the average tensile strain at failure of the different asphalt mixtures. The effects of

TABLE VII

AVERAGE TENSILE STRAIN AT FAILURE OF LABORATORY SPECIMENS (10^{-3} in./in.)

Asphalt Pen. Grade	91 pen.			124 pen.			160 pen.		
Asphalt Content, %	4.5	5.0	5.5	4.5	5.0	5.5	4.5	5.0	5.5
a. Temp. = +20°C	10.497	8.175	8.332	9.659	7.685	8.017	9.341	8.358	8.617
b. Temp. = 0°C	3.028	3.039	3.617	3.399	4.057	4.119	3.579	3.627	4.248
c. Temp. = -10°C	4.231	4.524	4.481	4.179	4.023	3.620	3.852	4.213	3.623

temperature, asphalt grade and asphalt content on the average tensile failure strain are illustrated in Fig. 30.

The analysis of variance indicated a strong evidence of temperature effect on the tensile strains at failure. The observed significance level was 0.0001. The average tensile strains at failure of all mixtures at 20°C were much higher than those at 0° and -10°C, respectively. This finding primarily reflected the elastic response of the asphalt mixture at relatively low temperatures. This elastic behavior would generally reduce the ability of the asphalt paving mixture to absorb the contraction strains developed at low temperatures. However, no significant differences were observed between the average tensile strains at 0° and -10°C, respectively.

The analysis of variance did not show evidence of asphalt grade differences in the tensile strain values and the observed significance level was greater than 0.1. No clear trend could be obtained from Fig. 30 between the asphalt grade and the average tensile strains at failure.

The effect of asphalt content on the measured tensile strain values was, apparently, dependent upon the temperature. For instance, average tensile strains at 20° and 0°C considerably increased as the asphalt content of the mixture increased from 5.0 to 5.5 percent by weight. On the other hand, maximum tensile strains at 20°C were associated with 4.5 percent asphalt content. Also, no particular asphalt percent could be reported as the optimum content for maximum tensile strains at -10°C.

It seems that the tensile strain results alone do not show a great amount of information concerning the behavior of these asphalt mixtures. However, it was hoped that the parameter of stiffness at failure, a parameter that combined both the tensile strength and the tensile failure

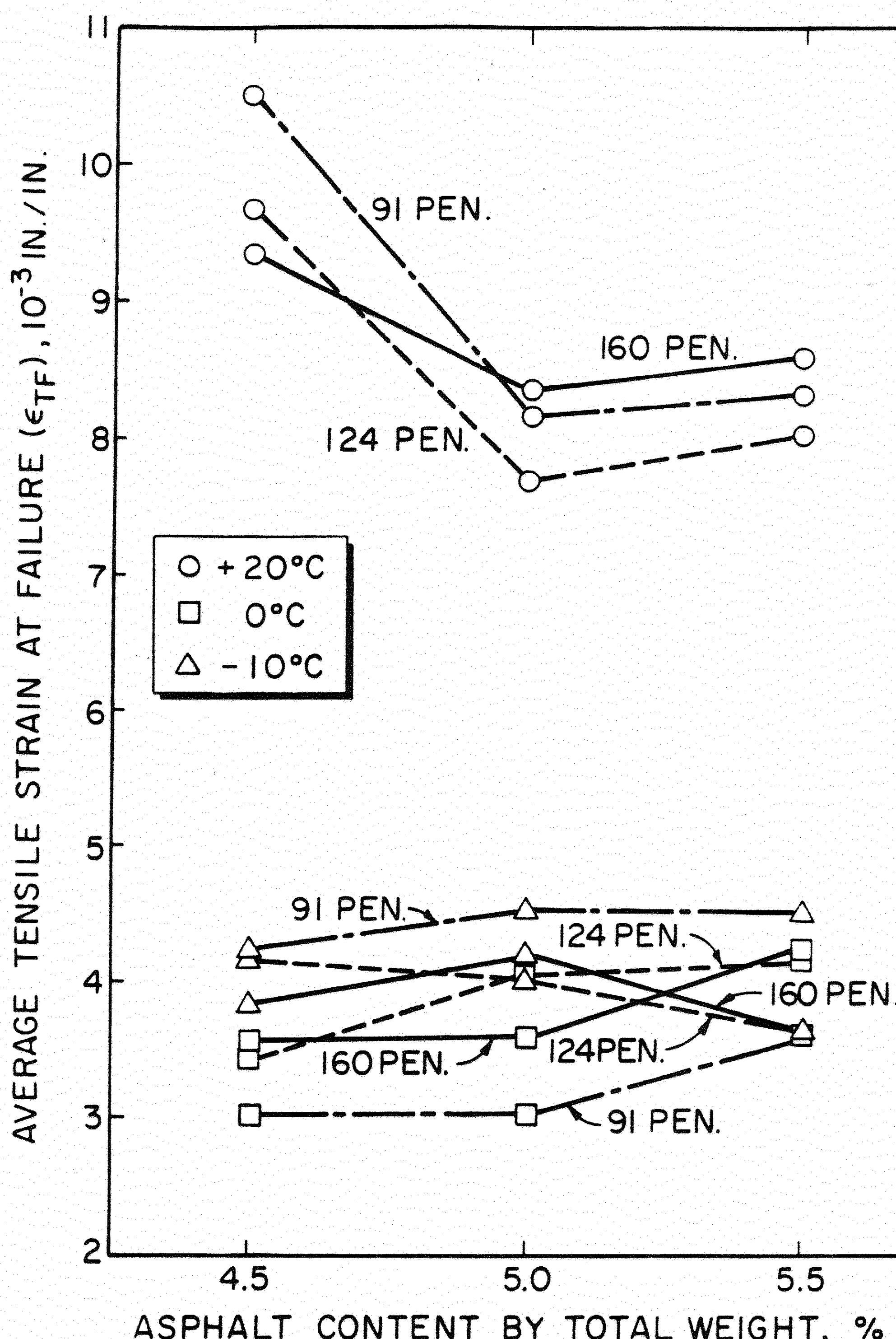


Figure 30. Average Tensile Strain at Failure Versus the Asphalt Content of the Mix

strain responses, would be a better indicator of the ability of a particular paving mixture to resist cracking at low temperature.

Ultimate Failure Stiffness: The average ultimate failure stiffness values are given in Table VIII. These values are also represented in Fig. 31 to show the effect of temperature, asphalt grade and asphalt content on the failure stiffness. Analysis of variance showed that temperature had a significant effect on the measured failure stiffness values. The observed significance level associated with this effect was 0.0001. As temperature decreased, failure stiffnesses considerably increased. Fig. 31 shows that the rate of increase was a function of both asphalt grade and asphalt content.

The results also indicated that high stiffness values were associated with low-penetration asphalt cements. At all temperature levels, failure stiffnesses significantly decreased as softer asphalt grades were incorporated in the mix. The observed significance level was less than 0.005 as indicated by the analysis of variance. Again, a significant interaction between temperature and asphalt grade was observed. This means that the effect of employing different asphalt grades on the failure stiffness values was not the same at each temperature level. For instance, the reduction in failure stiffness values with the use of soft asphalt grades (124 and 160-penetration asphalts) was more pronounced at 0° and -10°C than at 20°C. As previously discussed, transverse cracking was attributed, in part, to the low-temperature response of the stiff asphalt in the surfacing mixture. Considering the remarkable decrease in failure stiffness at low temperatures associated with the soft asphalt grades, it appears that transverse cracking in Oklahoma could be either eliminated or reduced by using relatively softer asphalt grades

TABLE VIII

AVERAGE ULTIMATE FAILURE STIFFNESS VALUES OF LABORATORY SPECIMENS (10^3 psi)

Asphalt Pen. Grade	91 pen.			124 pen.			160 pen.		
Asphalt Content, %	4.5	5.0	5.5	4.5	5.0	5.0	4.5	5.0	5.5
a. Temp. = +20°C	5.685	8.809	7.739	5.181	8.014	5.859	2.647	3.464	3.378
b. Temp. = 0°C	81.177	80.759	65.142	67.556	53.064	45.663	52.469	42.568	31.868
c. Temp. = -10°C	90.313	86.497	86.852	85.761	85.576	81.029	83.155	75.871	80.217

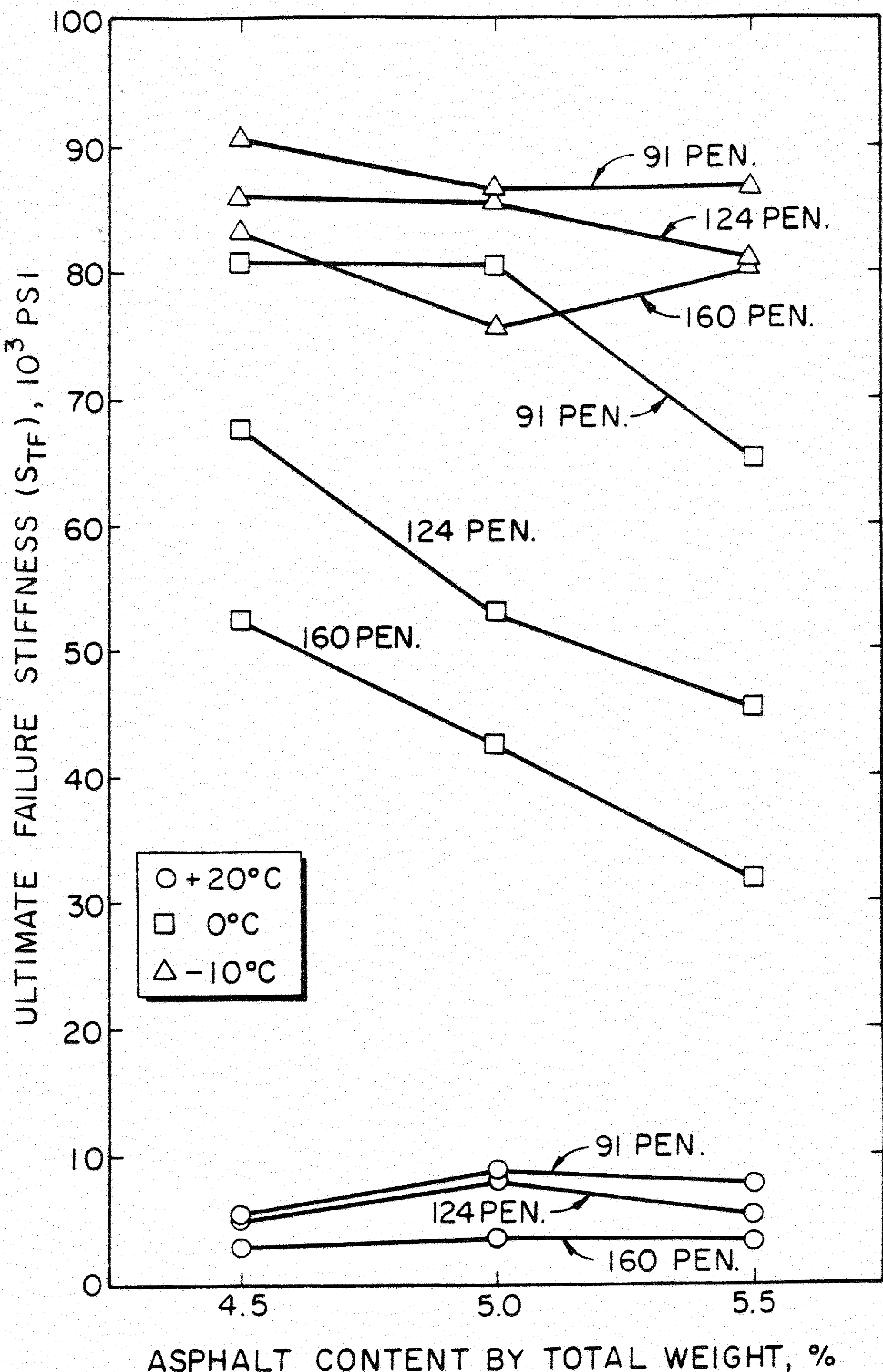


Figure 31. Average Ultimate Failure Stiffness Versus the Asphalt Content of the Mix

in surface course mixtures.

Asphalt content showed a significant effect on the failure stiffness values. The observed significance level was less than 0.005. However, this effect varied considerably between the three temperature levels. At 20°C, mixtures with 5.0 percent asphalt showed the highest ultimate failure stiffness values. At 0° and -10°C, failure stiffnesses gradually decreased, with the exception of the 160-penetration asphalt mixtures at -10°C, as asphalt content increased. This indicates that asphalt mixtures with relatively high asphalt contents may have greater ability to resist transverse cracking at low temperatures.

Stiffness Moduli of Laboratory Asphalt Cements and Mixtures

McLeod's nomographic procedure was used to calculate the stiffness moduli of the fresh asphalt cements and mixtures at various low temperatures. For comparison purposes, a 5.0 percent asphalt by weight was employed to estimate the stiffness moduli of the various asphalt concrete mixtures. This amount of asphalt was the approximate optimum content for the mixtures prepared with the 91-penetration asphalt cement grade. Table IX and X summarize the average stiffness moduli of the asphalt cements and asphalt mixtures, respectively. Also, Figs. 32 and 33 provide plots of these values versus the corresponding service temperatures.

Data obtained from this analysis clearly indicated the significant effect of service temperature on the performance of asphalt cements and mixtures. As can be seen in Figs. 32 and 33, stiffness moduli of all asphalt binders and mixtures vastly increased as service temperature decreased from 0° to -30°C. The stiffness modulus at a given temperature

TABLE IX

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 X

AVERAGE STIFFNESS MODULI OF LABORATORY ASPHALT MIXTURES

Asphalt Penetration at 25°C	Volume Conc. of Agg. Mix ($\grave{C}_v$)	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

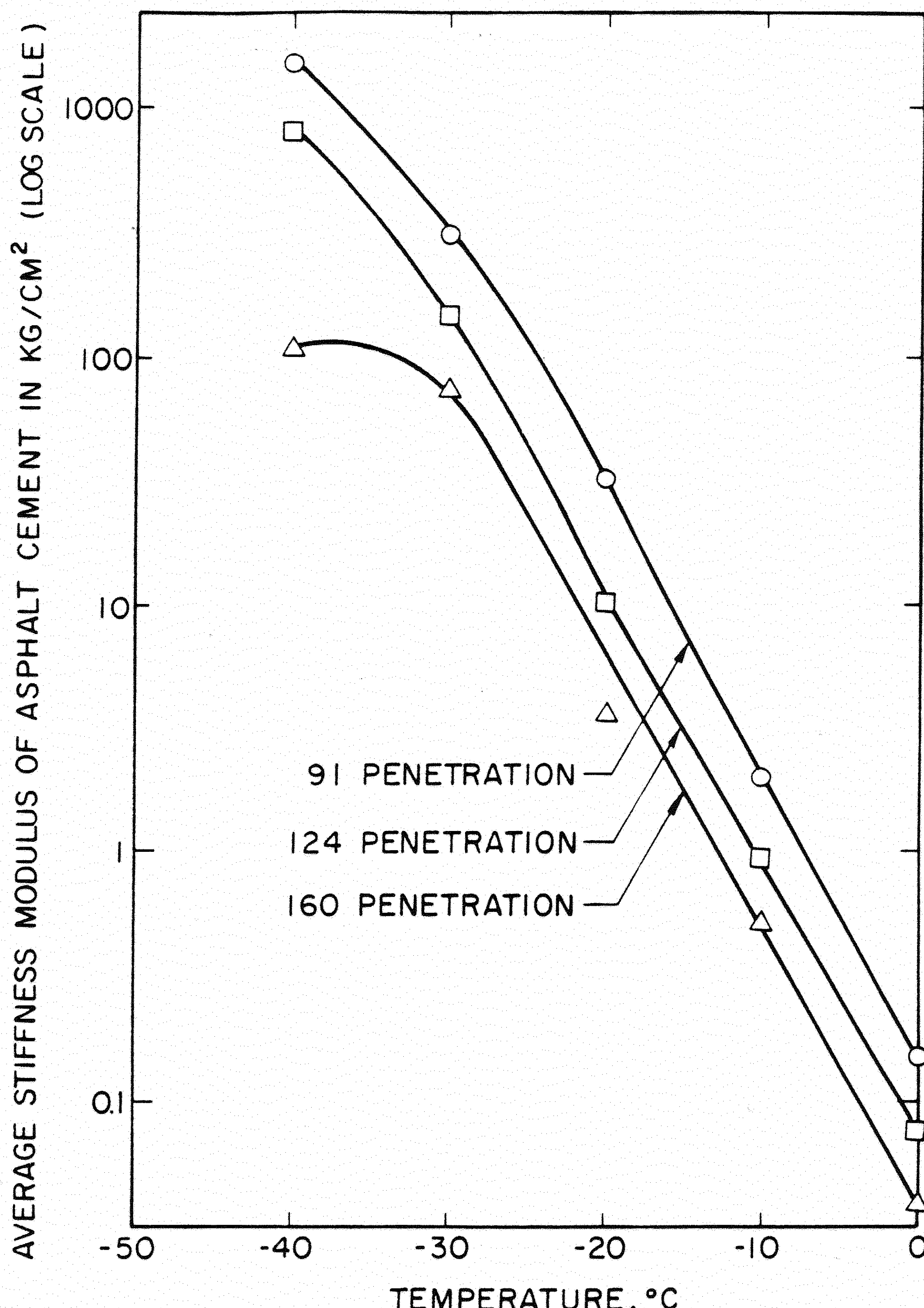


Figure 32. Stiffness Moduli of Laboratory Asphalt Cements Versus Service Temperature

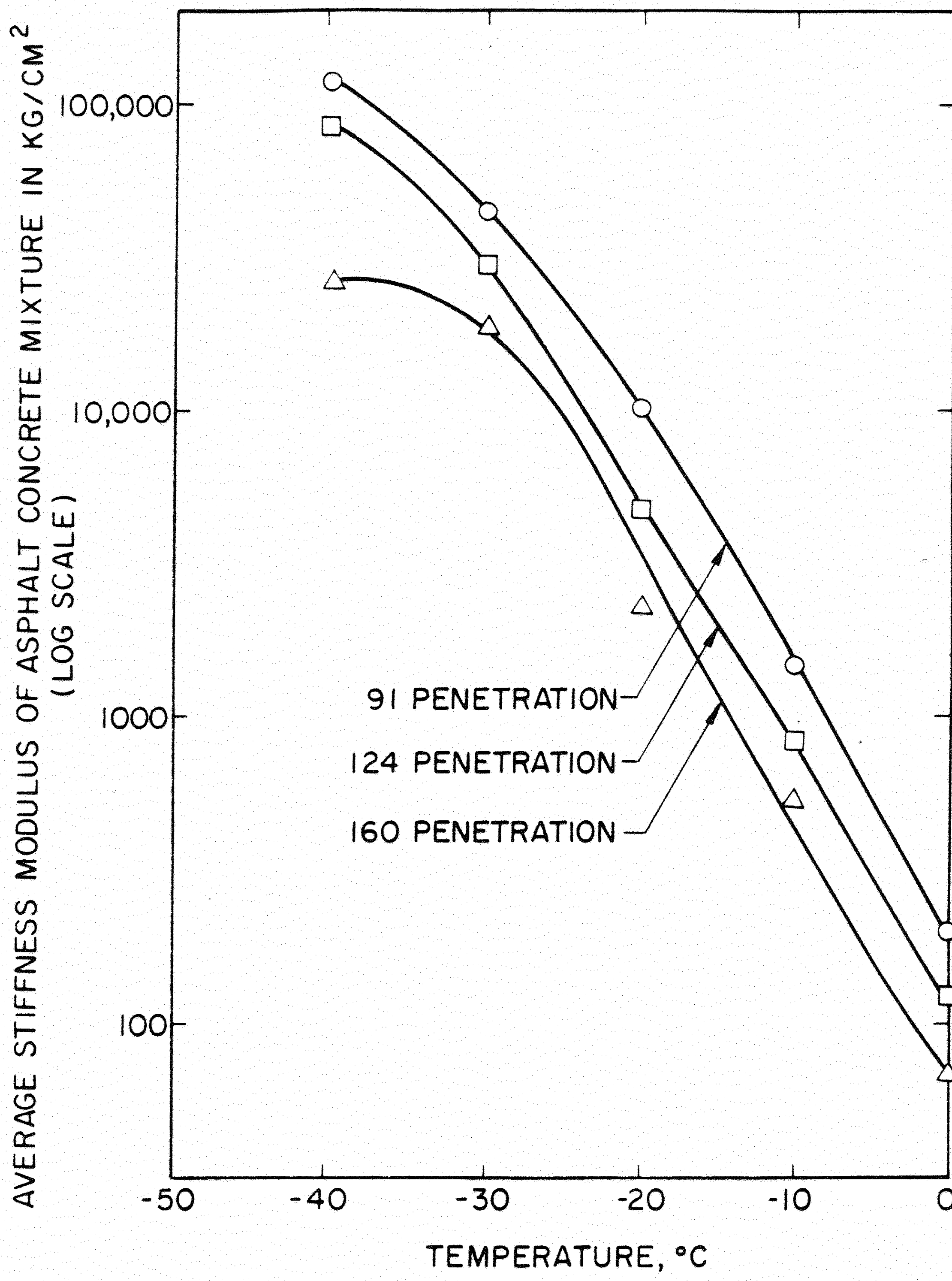


Figure 33. Stiffness Moduli of Laboratory Asphalt Concrete Mixtures Versus Service Temperature

is apparently a function of the asphalt grade. For example, values given in Table IX show that the stiffness values for the 91-penetration asphalt cement are significantly greater than those of the 124 and 160-penetration grades. Also, as temperature decreases from 0° to -30°C, the stiffness modulus of the 91-penetration asphalt cement increases at a rate twice that of the 124-penetration grade and four times that of the 160-penetration grade. Below -30°C, the stiffness modulus of the 160-penetration asphalt cement slightly increases while the stiffness moduli of the other two grades show a rather great increase. These findings point out the sensitivity of asphalt cements to low-temperature changes and show that low-penetration (high viscosity) asphalt cement grades will generally exhibit significantly higher stiffness moduli at these low temperatures.

As reported earlier for the field samples, a satisfactory correlation between the stiffness moduli of field paving mixtures and the observed degree of cracking was found. Previous studies indicated that the ability of a particular paving mix to resist cracking may be predicted from the stiffness modulus of the mix at the expected minimum service temperature. Therefore, the stiffness values of the various asphalt mixtures employed in this study were compared to the limiting stiffness values shown in the literature for the same conditions of temperature and rate of loading (23).

From Fig. 33, the stiffness modulus of the first mixture group employing the 91-penetration asphalt is approximately 17,783 kg/cm^2 (253,000 psi) at -23.3°C. Compared with McLeod's design guide, this value is considerably above the critical limit specified for -10°F (-23.3°C). According to McLeod's criteria, this mixture should not be used in pavements subjected to a minimum service temperature of -10°F, if transverse cracking is to be avoided. Consequently, this mixture is considered to be much too stiff to eliminate transverse cracking at temperatures lower than -10°F.

The stiffness moduli at -25° and -40°F (-31.7° and -40°C) of the second mixture group (124-penetration asphalt mixtures) are approximately 37,154 and 84,370 kg/cm^2 (528,448 and 1,200,000 psi), respectively. Again, these values are higher than the maximum safe stiffness moduli recommended to eliminate transverse cracking at these temperatures. On the other hand, the stiffness moduli of the third mixture group prepared with the 160-penetration asphalt cement is considerably lower than those suggested by McLeod. Apparently, surface course mixtures with this grade of asphalt could eliminate cracking in pavements subjected to temperatures as low as -40°F (-40°C).

As previously discussed, a minimum temperature of -10°F (-23.3°C) could be expected to occur at a pavement depth of 2.0 in. in Oklahoma. Based on the previous analysis, the 85-100 penetration asphalt cement commonly used in paving mixtures in Oklahoma seems to be too hard to avoid transverse cracking at this temperature. This analysis also indicated that transverse cracking at -10°F could be eliminated if the 120-150 asphalt penetration grade is used.

It should be pointed out that results achieved with relatively soft grades of asphalt cement were found very satisfactory and transverse cracking was practically eliminated without sacrificing pavement stability at high summer temperatures in Canada (23). Only a slight modification was required in the Canadian laboratory mix design procedures to achieve higher densities. This modification was suggested to reduce the tendency of these paving mixtures to densify much more rapidly under traffic. If high temperature stability requirements can not be met with higher penetration asphalt cements and standard O.D.O.T. surface mixture gradation, a less desirable asphalt, i.e., an asphalt having a higher stiffness modulus, will have to be selected and some cracking tolerated.

CHAPTER VII

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The major conclusions drawn from this study may be summarized as follows:

1. Examination of recently developed transverse cracks revealed that, in most cases, the cracks had originated at the pavement surface. Thus, the major cause of these transverse cracks appears to be cold-temperature contraction of the asphalt concrete surface layer. This contraction of the surface is restrained by the underlying layers. However, some of these transverse cracks extended through the pavement matrix and may have originated at the pavement-subgrade interface due to thermal stresses, traffic stresses, or various problems in the subgrade material.

2. The tensile splitting test appears to be a practical method for evaluating the tensile properties of asphalt concrete mixtures and warrants inclusion as a routine test in future design procedures that take into account the mixture properties and behavior at low as well as high temperatures.

3. Temperature had a highly significant effect on the measured tensile properties of both field core samples and laboratory asphalt mixtures. As temperature decreased, tensile strengths (tensile stresses at failure) and failure stiffnesses remarkably increased and tensile strains at

failure decreased. This is primarily due to the increase in stiffness of the asphalt binder.

4. A satisfactory correlation was found between the tensile splitting test results of the field core samples and the observed degree of pavement cracking. The occurrence of transverse cracking was found to increase as the failure strains decreased and the failure stiffnesses increased.

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

6. Lower tensile strength (tensile stress at failure) and ultimate failure stiffness values were observed for laboratory asphalt mixtures when softer asphalt grades were incorporated in the mix. However, no clear trend was found between average tensile strains and asphalt viscosity or penetration.

7. Increasing the asphalt content of the mix reduced failure stiffness and tensile strength in most cases. Again, tensile strains at failure did not significantly differ as asphalt content increased.

8. Based on the limiting stiffness concept, the 85-100 penetration asphalt commonly used for paving operations in Oklahoma is considered too hard a grade to avoid transverse cracking at the minimum service temperature to be expected in Oklahoma.

Recommendations

The following recommendations are pertinent to the results and the observations obtained from this research investigation:

1. Future research should be directed toward studying the effects of temperature cycles, traffic load stresses and subgrade properties on transverse cracking initiation and propagation.

2. The tensile splitting test should be utilized in further comprehensive testing of cracked and uncracked pavement sections to develop "safe" or "critical" strength, strain and/or failure stiffness values. The ability to resist transverse cracking of a particular paving mixture, which is satisfactory in all other respects, could then be predicted by comparing its tensile splitting test results with the developed design limits.

3. Current asphalt cement specifications are primarily concerned with the rheological properties at moderate and high temperatures (penetration at 77°F, viscosity at 140° and 275°F and softening point). For a wide range of design situations, the viscosity or some other consistency measurement at low temperatures should be included in future specifications.

4. The effect of using an asphalt cement with a lower stiffness modulus, either a 120-150 penetration asphalt or a 85-100 penetration asphalt with improved low-temperature sensitivity,and the corresponding modifications required in current mix design procedures should be investigated with regard to high-temperature stability problems. Based on the results of other research studies, the low-temperature sensitivity of asphalt binders could be improved by additives such as synthetic polymers, natural rubber and asbestos fibers.

5. A more comprehensive and objective crack surveying technique for the Oklahoma highway network is needed. This technique should also include a measure of pavement serviceability.

6. Future research investigation is required to determine the most practical, efficient and economical method of treating existing cracked pavements.

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APPENDIX A

TEST SITE INFORMATION

TABLE XI

TEST SITE INFORMATION

Test Site Number	Highway Number	Location	County	Construction Project Number	Date Completed	Overlay Date
1	US 177	4.0 miles south of US 66	Lincoln	S 251 (9) S	1951*	1972
2	US 177	8.0 miles south of US 66	Lincoln	S 251 (10) S	1951*	1972
3	US 177	4.5 miles south of US 62	Pottawatomie	S 251 (6) (7)	1951*	1971
4	US 177	6.5 miles south of US 62	Pottawatomie	S 251 (6) (7)	1951*	1971
5	IS 35	4.0 miles north of Cimmaron River Bridge	Payne	I-35-4 (19) 165	1961	....**
6	IS 35	4.0 miles north of Perry	Noble	I-35-4 (29) 187	1963	1971
7	IS 40	1.0 mile east of Oklahoma-Pottawatomie county line	Pottawatomie	I-40-5 (49) 193	1965	1972
8	IS 35	5.5 miles north of Cimmaron River bridge	Payne	I-35-4 (19) 165	1961	....**
9	IS 40	2.5 miles east of Oklahoma-Pottawatomie county line	Pottawatomie	I-40-5 (49) 193	1965	1972

* These projects were completed before 1951.

** These projects have had no overlays.

TABLE XII

TRANSVERSE CRACKING DATA

Test Site Number	No. of Cracks in the chosen 500 ft length				Cracking Index*
	Multiple	Full	Half	Part	
1	--	1	11	52	6.5
2	--	10	1	3	10.5
3	1	7	15	14	15.5
4	1	9	29	27	24.5
5	--	--	4	7	2.0
6	--	4	10	29	9.0
7	--	19	2	--	20.0
8	--	--	--	--	0.0
9	--	--	1	3	0.5

* see equation (2.1)

APPENDIX B

PROPERTIES OF LABORATORY MATERIALS AND MIXTURES

TABLE XIII

SUMMARY OF SIEVE ANALYSIS TEST RESULTS

Sieve Size or Number	Per Cent Passing by Weight, %			
	1/2-in. Crushed Limestone	Limestone Screenings	Coarse Sand	Fine Sand
1/2-in.	100.0	100.0	100.0	100.0
3/8-in.	82.0	100.0	99.0	100.0
4	9.0	96.0	92.0	100.0
10	1.0	58.0	71.0	99.0
40	1.0	28.0	18.0	70.0
80	1.0	21.0	3.0	16.0
200	1.0	14.6	0.8	6.3

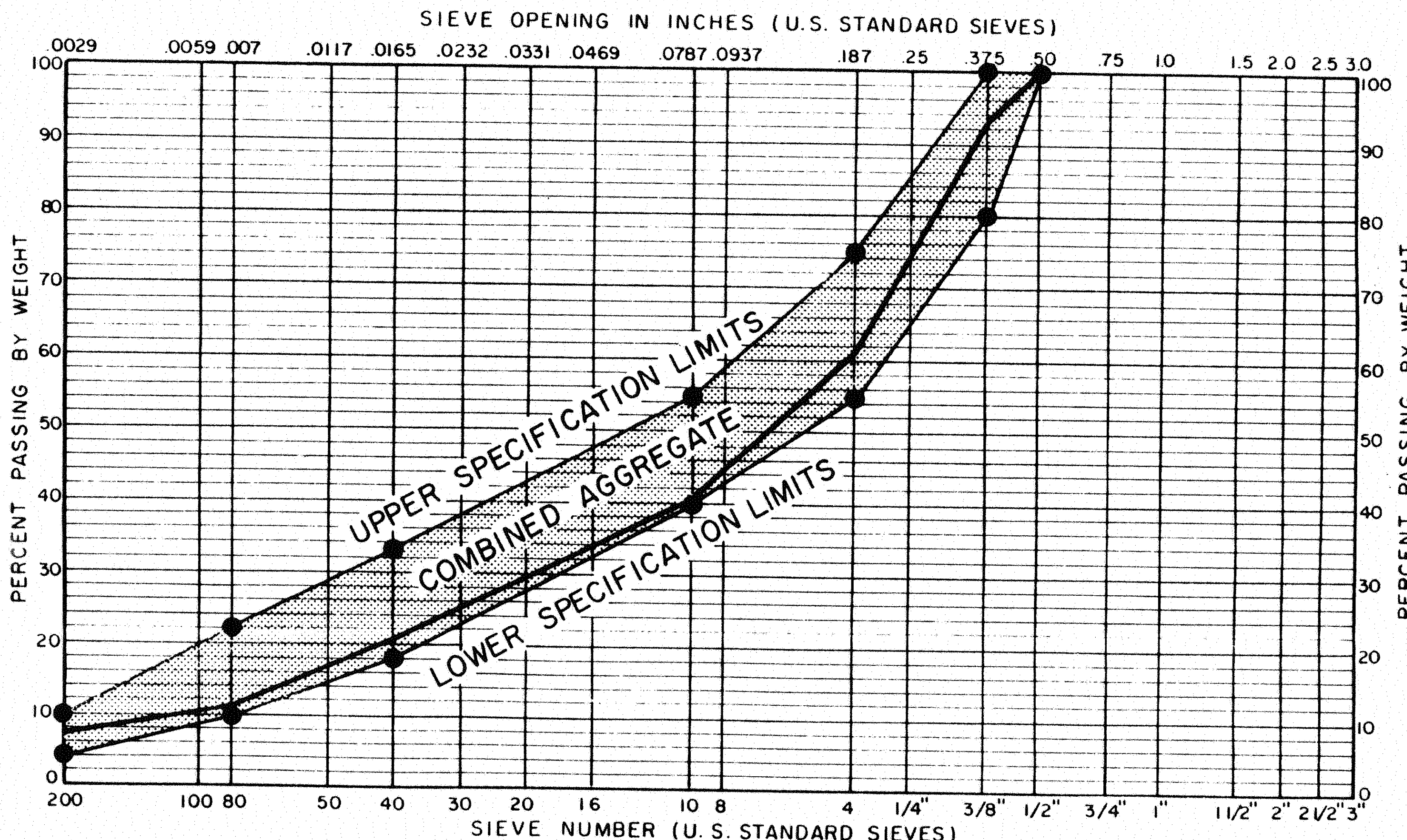

Figure 34. Combined Aggregate and Specification Limits Grading Curves

TABLE XIV

SUMMARY OF SPECIFIC GRAVITY AND WATER ABSORPTION TEST RESULTS

Aggregate	Per Cent by Weight	Bulk Specific Gravity	Apparent Specific Gravity	Percent Absorption, %
1/2-in. crushed Limestone	40	2.741	2.871	1.656
Limestone screenings	40	2.716	2.867	1.941
Coarse sand	10	2.605	2.662	0.820
Fine sand	10	2.576	2.669	1.376
Combined Aggregate	100	2.700	2.828	1.658

TABLE XV

RHEOLOGICAL PROPERTIES OF ASPHALT CEMENTS

Sample Number	1	2	3
Specific gravity, 77°F/77°F	1.004	0.999	0.997
Penetration 200 gm, 60 sec, 39.2°F	26.000	34.000	44.000
Penetration 100 gm, 5 sec, 77°F	91.000	124.000	160.000
Absolute Viscosity 140°F, 300 mm Hg vacuum poise	1485.000	939.300	652.300
Kinematic Viscosity 275°F C. Stokes	3876.000	325.000	268.800
Ring and Ball Softening point, F°	112.060	107.240	102.740

TABLE XVI

PROPERTIES OF COMPACTED ASPHALT MIXTURES

Asphalt Penetration	Asphalt Content by Weight, %	Bulk Specific Gravity	Maximum Specific Gravity	Per Cent Density	Hveem Stability, lb
91	4.5	2.423	2.588	93.62	50.6
	5.0	2.458	2.574	95.49	47.6
	5.5	2.474	2.554	96.87	45.9
124	4.5	2.420	2.580	93.80	55.5
	5.0	2.454	2.564	95.71	50.3
	5.5	2.455	2.536	96.81	47.7
160	4.5	2.420	2.574	94.02	50.3
	5.0	2.451	2.563	95.63	48.4
	5.5	2.453	2.541	96.54	47.0