STIFFNESS EFFECTS ON FATIGUE LIFE OF ASPHALTIC CONCRETE

by

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ABSTRACT

The study reported herein was concerned with the effect of stiffness on fatigue resistance of asphaltic concrete. The factors believed to contribute significantly to stiffness are temperature, support pressure and asphalt content. These are examined in the study.

An extensive survey of the literature on fatigue testing is given, and the merits of a relatively new test equipment--the deflectometer--are observed. This equipment was used in the study. Test procedures are adequately described.

The aggregate gradation used was very coarse and gap-graded and resulted in a stiff mix. Results of the test are not, therefore, in absolute conformity with data reported from other mixes or test procedures.

CHAPTER 1

INTRODUCTION

Design considerations for asphaltic concrete pavements aim at providing a pavement surface that will give good service to traffic throughout its service life. To this end the pavement is designed for stability, durability, flexibility, imperviousness and skid resistance. Other factors which are also considered are fracture (tensile) strength and fatigue resistance.

Stability or resistance of a mix to deformation is affected by frictional resistance within the mixture, cohesion, and inertia. Major contributing factors are aggregate characteristics (especially surface texture), compaction of mix, and asphalt content. For good stability the asphalt content must be maintained at a comparatively low value.

Durability is the resistance of the mixture to weathering, including aging, and to the abrasive action of traffic. This is normally achieved through use of high asphalt contents, densely graded aggregate, and well compacted mixtures.

Flexibility is defined as the ability of the mix to conform to variations in subgrade and base elevations. It

is promoted by high asphalt content and comparatively open-graded aggregate.

Imperviousness is desirable if the pavement structure is to be durable. Permeability is here defined not only in terms of percolation of water into the pavement but also of air and water vapor, both of which are detrimental to the mixture. Skid resistance is furnished by the same factors that contribute to stability. It also requires aggregate type that is resistant to the polishing action of traffic.

Fracture or tensile strength is the maximum strength of the mixture. This is important when considering heavy loads, particularly at low temperatures or when the subgrade is saturated. It is influenced by asphalt content, aggregate gradation, mineralogical composition of fine aggregate, and mixture density. Increase in asphalt content and mineral filler tends to increase fracture strength, within limits.

Flexibility of asphaltic concrete pavements has also been referred to as fatigue resistance (1). In its strictest sense, however, fatigue failure refers to cracking of the asphaltic concrete resulting from the resilient or elastic deformation. This may be associated with little or no permanent deformation of the pavement at the outset. Data seem to indicate that all mixtures exhibit fatigue

(2), and that cracking is related to the intensity of the tensile stress induced in the mixture. Mixture variables that contribute to high fracture strength would therefore cater to fatigue resistance.

It will be seen from this brief survey that there is a conflict of interest between factors contributing to stability and skid resistance on one hand, and those required for durability and flexibility on the other. A balance has thus to be struck between a stable but rough and weathered surface, and a smooth, otherwise flexed surface. As to which side gets the upper hand is largely a question of economics and engineering "judgment."

While this conflict is being resolved fatigue resistance is unwittingly being involved. We noted earlier that fatigue, which is a process of progressive localized permanent structural change due to fluctuation of stresses and strains (2), is influenced by asphalt content among other things. Hence the ultimate design will have a direct contribution on the fatigue life of the pavement.

Fatigue failures of asphaltic surfaces were noted and commented on by Hveem and Carmany (3) in 1948. In a later paper, Hveem (4) presented data on measured pavement deflections and performance, and suggested limiting deflections for different types of pavements. The implication to this was that a pavement could sustain any number of load

repetitions provided the permissible deflection was not exceeded.

Following on Hveem's lead many researches have been carried out in the last two decades, notably in Europe and the United States. These have involved test roads such as the WASHO and the AASHO, and laboratory tests on both specimens taken from the pavement and specimens made in the laboratory. The laboratory investigations which have been undertaken are of two basic approaches. The first approach is concerned more with the fundamental behavior of the asphaltic concrete as an engineering material. Tests are carried out with established methods of fatigue testing such as the rotating cantilever machine used by Pell (5). The second approach endeavors to simulate field conditions. It includes such notable works as reported by Monismith (6), Papazian and Baker (7), Monismith, Secor and Blackmer (8), Jimenez (9) and Vallerga, Finn and Hicks (10) to mention only a few. These will be discussed later in the review of literature.

Of particular interest here is the apparatus and test procedure that was reported in Jimenez (9). Commenting on the rationale behind this test method Jimenez notes that (p. 4):

> In the past, the usual procedure used in testing for flexibility has been to load a prismoidal shaped specimen in some sort of beam condition. This type of testing does

not appear to be adequate for duplicating service conditions, although it does serve to obtain comparative values of parameters with which to determine the effects of certain variables. A model for testing, more closely related to the conditions in the pavement, is a large diameter disc fixed about its periphery and loaded on its center.

To simulate the effect of subgrade reaction the deflectometer, as it is now called, has a reaction unit which can be adjusted to give variable support to the pavement model. More description of this will appear in the report. Test data obtained with this procedure has been reported by Jimenez (9, 11), and Layman and Philippi (12). It indicated that the deflectometer yields reproducible results and that it is sensitive to factors affecting the flexibility and fatigue resistance of asphaltic concrete.

Laboratory fatigue tests are generally of two types: controlled-stress where the load is in the nature of alternating stress of constant amplitude, and controlled-strain where loading is in the form of alternating strain of constant amplitude. Modes of failure of the two types are different. In the former a flaw will lead to stress concentration at the point of initiation, which will in turn lead to fast propagation and complete failure or rupture. In the latter, however, cracking will result in a decrease in stress and hence a slower rate of propagation. The fatigue life measured, therefore, includes a considerable

length of time required to initiate a crack that defines failure. In fact, failure in this case is arbitrarily determined since complete rupture will take a long time to achieve.

The choice as to which mode of testing should be adopted depends on the type of pavement. It has been suggested (10) that since thin layers of asphaltic concrete exhibit greater flexibility under load application they should be made with mixes of low stiffness. Consequently, from available fatigue data, the appropriate mode of test would be controlled strain. On the other hand, thick pavement sections impart more stiffness to the structure and hence require stiffer mixes. Testing procedure appropriate to this would be controlled stress.

In actual field performance, pavements are subjected to nearly constant load during their service life. This makes the controlled-stress mode of testing a more logical one.

The deflectometer is essentially a controlled-stress type of apparatus. The same load magnitude is maintained during the period of testing. This is made possible by using a floating load system.

While the work reported by Jimenez (9) was extensive, the primary object was to evaluate the deflectometer as a fatigue testing machine. The material used was limited

to fine-grained asphaltic concrete mixtures, and it was suggested that coarse-grained mixtures should also be tested to find out whether they conformed with the general findings. The work by Layman and Phillipi (12) was concerned with flexural properties of certain black base mixtures. Among the things investigated was variation of the mixture stiffness with temperature for a constant asphalt content.

The work reported here is an extension of the two works above, and deals specifically with the effects of stiffness on the fatigue resistance of the asphaltic concrete mixtures. The materials and aggregate gradation coincide with those used on the construction of the U.S. Interstate Highway 19. Details of this will be discussed in the report.

Stiffness or dynamic stiffness, as it is often called, has been defined simply as the ratio of stress amplitude to strain amplitude (2, 4, 5). It is dependent on both the time-of-loading and the test temperature, and gives a reflection of the rheological behavior of the mixture. Stiffness of asphaltic concrete pavements is a function of all the components that constitute the pavement, viz., subgrade, sub-base, base and surfacing. However, Hveem (4, p. 46) prefers, for design purposes,

> To associate the concept of stiffness with the pavement and base structure alone in which

> case there will not be a consistent relationship between stiffness and deflection as the character of the supporting soil will then represent a variable: "resilience."

In this study the subgrade support is a contributing factor to stiffness. The effect of initial support on the stiffness and/or fatigue life of the mixture forms part of the investigation. Other variables are temperature and asphalt content.

The significance of mixture stiffness in the fatigue performance of pavements has been recognized from the earliest investigations, although disparity exists between the interpretation then (13, 14), and what is currently known. Every investigator after Rader (13) has made considerable contributions to this subject. Finn (2, p. 66) has the following to say:

> To manipulate the fatigue characteristics of asphaltic mixes, it would be desirable to have a single parameter whose association with fatigue is known. At present the stiffness modulus--seems to be most logical, inasmuch as a considerable amount of information has been reported relative to its correlation with fatigue life--.

Yet we do not seem to have reached the ultimate, or even approached it, because although the knowledge is piling, pavements continue to fail in fatigue. This is hardly surprising when it is recalled that stiffness and fatigue resistance are functions of many indeterminate variables including the mixture properties and environmental conditions.

Also, some of the basic assumptions of homogeneity isotropy and elasticity assumed in the analysis are only close approximations at best. Therefore, any additional information in this field, especially by test methods that approach site conditions, will be a welcome contribution. It is hoped that this study will add to the knowledge.

CHAPTER 2

LITERATURE REVIEW

One of the earliest investigations into the cracking resistance of asphaltic concrete was reported by Rader (13) in 1936. This was largely concerned with the physical properties of the mixture at low temperatures. Beam specimens 2" X 1½" X 6" were cut from two existing pavements of contrasting performance (one heavily cracked and another in good condition) and tested in flexure. Asphalt was also extracted by the Abson method and consistency tests performed on it. Remolded samples were tested for stability. The results of this investigation are of particular interest: Rader reports that the pavement which was in good condition after 8 years of service had average values of modulus of elasticity* of 479,000 psi, while the cracked pavement had average values of 624,000 psi. He noted that the great differences in stiffness were due to great differences in density, and that the high stiffness was responsible for

*The term "modulus of elasticity" used here is the same as "stiffness modulus" used elsewhere and defines the relationship between stress and strain for static loading. For dynamic loading the term "dynamic stiffness modulus" has been used. All these terms have the same connotation. The higher the modulus, the stiffer the mix.

high tensile stresses which resulted from low temperatures. Thus, he concluded that a low modulus of elasticity was desirable from the standpoint of reducing cracking. In a subsequent report (14) he produced more data which confirmed his initial findings. Although these findings do not seem to agree with our current knowledge, they paved way for laboratory and field research into flexural properties of asphaltic concrete pavements. It is now generally acknowledged that thermal stresses are not a main cause of surface cracks.

The first repeated-loading test was reported by Thomas (15) in 1948. His studies involved specimens 16 inches in diameter supported on a spring base and loaded centrally. Eight repetitions of load were applied on either side of the specimen. The strength of the specimen was evaluated from the amount of damage resulting from the load application. It was concluded that high stiffness was not advantageous since cracking occurred more on the stiffer mixtures.

The next work in line is that reported by Hveem and Carmany (3) and later extended by Hveem (4). In the latter work Hveem discussed the different components of a pavement and how they interact with each other during the service life of the pavement. Deflection measurements were made at different elevations in the pavement and road basement and

these were correlated with field performance. Comparison was also made with the results of the Road Test One-Maryland and the WASHO road test and it was concluded that a limiting deflection of about 0.020 inch was essential for repeated-flexure endurance. Hveem also presented initial results of laboratory fatigue tests on beams 2" X 2" X 10" cut from existing pavements. The test apparatus had a cam arrangement operating at speeds to simulate truck traffic. The results indicated that fatigue life decreased as the strain or deflection increased. Thus, concluded Hveem (4, p. 72),

> There is an unusually close correlation between observations of cracking and fatigue type failures--- and measured deflections---. These deflections appear to be associated with compression and rebound in the soil---. It may be said that the principal destructive force is the energy stored in the subgrade by each passing wheel.

Hughes and Faris (16) worked on the deformation of asphalt and sand mixes employing two rates of deformation. The results for high deformation rate (3 inches per second) were not very significant as they did not show any effect on mix variables. However, for the low rate of 0.07 inch per hour effects of asphalt consistency and temperature were significant in mixture deformability. Specifically, it was noted that failure occurred at lower magnitudes of deformation, and these varied with asphalt consistency and temperature susceptibility.

Wood, Goetz and McLaughlin (17, 18) presented papers on the strength of bituminous mixtures under different loading conditions. Tests were carried out on cylindrical specimens, loaded axially, and under different temperatures and loading pressures. The results showed a logarithmic relationship between deformation and load repetitions to failure. It also indicated a logarithmic relationship between temperature and compressive strength.

Several reports have been presented by Monismith et al. (6, 8, 19, 20, 21) on asphalt mixture behavior in repeated flexure. In Reference 6 Monismith presented some of the factors which should be considered in the analysis of flexibility characteristics of pavements. These included (a) properties of materials, that is, asphalt, aggregates and the mixture, (b) structural effects including slab thickness and load distribution, and (c) external conditions such as temperature, resilient properties of the subgrade, and magnitude, frequency and duration of the load. Detailed discussion of each of these factors was given. In particular he pointed out the significance of asphalt viscosity to the overall performance of the mixture, and how repeated stressing can cause strain-hardening and eventual cracking of the asphalt-fine aggregate matrix. In this study beam specimens 3" X 2" X 12" were compacted by a kneading compactor and tested in the manner of Thomas (15).

Significant conclusions of this study were that (a) the modulus of rupture increased with increase in asphalt content and the number of load applications, and (b) decreased temperature increased the modulus of rupture which was accompanied by increased rigidity under dynamic loading. In the paper cited in Reference 8 the discussion centered more on the effects of frequency of load application and stress reversal. Three rates of applications of 30, 15, and 3 per minute, with load application time of 1 second, were used. These corresponded with slow moving traffic on an urban highway operating at capacity. Monismith et al. concluded from this investigation, that for the range of frequencies and temperatures utilized, the frequency of loading had little effect on the modulus of rupture. The effects of stress reversal were also negligible; as long as the maximum strains for the reversal and non-reversal test-beams were the same. Based on this and other data available at the time, it was suggested (8, p. 197) that an "excellent criterion for prediction and prevention of fatigue failure" was the limiting bending strain. For field performance it was suggested that the radius of curvature of the loaded area would be a good indicator.

In presenting their work on flexural fatigue Deacon and Monismith (20) re-defined the factors affecting fatigue behavior both for field performance and laboratory testing.

Laboratory variables were categorized into (a) load variables such as pattern of stressing, time distribution of loading and mode of loading (whether simple or compound), (b) environmental variables, including temperature, moisture and aging, and (c) mixture variables. Significant variables in the mixture were aggregate type and gradation, binder type and hardness, and specimen air voids, surface texture, shape, size, stiffness, anisotropy and asphalt content. Emphasis was placed on compound-loading fatigue behavior since vehicular stresses were essentially compound in nature. The test equipment used was therefore designed for such loading. During each load application the specimen was flexed for a given duration after which it was forced to return to its original position. This resulted in stress reversal without corresponding strain reversal. Several conclusions were made from the study, but the most significant here was that (20, p. 12):

> In general it appears that load, environmental and mixture and specimen variables tending to increase the stiffness moduli of asphalt specimens--tend also (a) to increase the observed service lives for controlled-stress loading at any given stress level, but (b) to decrease the observed service lives for controlled-strain testing at any given strain level.

The stiffness modulus was found to alter considerably during the final phase of the test, due to the damaging fatigue loading.

The last paper (21) deals with prediction of pavement deflections from laboratory tests. A series of repeated-load tests were conducted on prototype pavement sections and observations were made on the behavior of the subgrade, base and surfacing. It was noted that temperature had a significant effect on the measured deflection. This effect was attributed mainly to changes in stiffness. In discussing the results, the authors stated that the resilient* characteristics of asphaltic concrete surfacing were dependent on the rate of loading and temperature. Nomographs presented by van der Poel in 1954, and modifications thereof by Heukelom and Klomp (22) gave good estimates of mixture stiffness, knowing the stiffness of the binder and volume of aggregate.

Another paper by Epps and Monismith (23) will be discussed later.

Papazian and Baker (7, 24) carried out fatigue tests on prismatic specimens tested as simple beams with an elastic support at the center. They reported that the endurance of the mixture was a function of the stress-level. In the second paper they analysed the effect of stiffness ratio on

*Deformations in pavement structures caused by moving vehicles are essentially elastic in the sense that they are almost completely recoverable. However, to avoid confusion the deflections have been termed resilient rather than elastic. This term as originally used by Hveem (4) referred to the subgrade.

pavement stresses, using Westergaards equations and Burmister's theory for two-layered systems. They noted, however, that maximum deflections depended not only on ratios, but also on the rigidity of the supporting subgrade.

Dehlen (25) presented a study of "chicken net" cracking* of a road surfacing, conducted in South Africa. Deflections were made by means of a Benkelman beam, and from the deflection profiles the curvature was determined. This was used to assess the stresses induced in the pavement. The author noted that a correlation existed between maximum deflection, curvature, and the condition of the surface. He stated that the curvature was dependent mainly on the modulus of the base material.

Although many of the laboratory test procedures so far tried to simulate subgrade reaction through the use of a spring base or an elastic central support, none had sufficient lateral confinement to approach field condition. The deflectometer reported by Jimenez (9, 11, 26), and Layman and Phillipi (12), had both devices incorporated. A subgrade support was provided by an oil-air system in a cylindrical chamber sealed at the top with a thin rubber membrane. The specimen was clamped onto the chamber by

*The terms "chicken net" cracking, "alligatoring" or "mapping" have been used to refer to the pattern of surface cracking typical of fatigue failure of asphaltic concrete.

means of a steel ring and bolts. A constant clamping force was maintained through the use of springs and metal spacer sleeves. A vibratory-kneading compaction method was developed along with this test procedure to compact specimens 4" and $17\frac{1}{2}$" in diameter. Further details of this will appear in the report. The general findings reported in Reference 9 were as follows:

(a) Fatigue life increased with increasing asphalt viscosity.

(b) A logarithmic relationship between applied stress, S, and fatigue life, N, resulted in the expression $S=I_oN^{-b}$, where I_o and b were mix constants. For normal asphaltic mixtures b was very close to 1/5.

(c) There was an optimum asphalt content associated with maximum fatigue life.

(d) There was an optimum asphalt content associated with maximum dynamic modulus.

(e) Dynamic modulus increased with increase in specimen density (for an optimum asphalt content).

(f) Within the limited range of stress and specimen density studied, the dynamic modulus for a mixture was relatively constant regardless of stress level or specimen thickness.

From these studies Jimenez suggested that fatigue life of asphaltic concrete could be increased by increasing the

density by compaction (within limits), increasing the asphalt content to an optimum value, increasing viscosity of asphalt (if loss of density is not great), or by increasing the dynamic modulus of elasticity with the use of mixture additives. The findings of Layman and Phillipi (12) confirmed the above results. In addition they reported that:

(g) Radial stresses within the specimen varied approximately in phase with the applied stress.

(h) For a given test temperature, as radial stress increases fatigue life decreases.

(i) The radial strain for a given thickness increased as test temperature increased, due to decrease in dynamic modulus at failure.

(j) Total deflection at failure increased at higher test temperatures.

Performance of flexible pavements has been analysed by Kondner and Krizek (27) based on the findings of the AASHO Road Test and a rigid bearing-plate study. Analytical methods of analysis were employed in evaluating stresses and effects of layers on deflections. It was noted in particular that temperature decrease had an effect of increasing the stiffness of the soil-pavement system. Also the load deflection response for deflection-controlled tests was different from that of load-controlled tests.

White (28) presented a summary of a comprehensive study on fatigue and deflections of asphaltic pavements which had been conducted by the Oregon State Highway Department. These consisted of field tests using the Benkelman beam, laboratory tests on beams cut from the pavements, and laboratory tests on molded specimens. One interesting deduction from the laboratory tests was that there was "no evidence of connection between void content and fatigue durability." From the field-cut beams, also, no correlation was found between the pavement characteristics and the number of deflections to failure.

An interesting study on paving materials for orthotropic steel plate bridges was given by Metcalf (29). This was in conjunction with the construction of such a bridge at San Mateo in the San Francisco Bay. Composite beam specimens 4" wide by 18" long consisted of a steel plate 5/8" thick and a pavement layer 1½" thick. Among the findings of the study was that a temperature of 72°F was fairly critical for flexural fatigue of asphaltic concrete. Fatigue life tended to increase at temperatures higher or lower than 72°F.

From France, Bazin and Saunier (30) submitted results of a study on deformability, fatigue and healing properties of asphaltic concrete. A specially designed apparatus imparted forced vibrations into a clamped testbar. The force

(load) and displacement were measured with transducer's and carrier frequency amplifiers. Specimens differed in shape and size. Some were parallelepipedic with dimensions 2 cm. X 2 cm. X 15 cm., while others were trapezoidal. The results showed the same type of correlation between stiffness modulus and temperature, time of loading, asphalt type, and void content, as had been reported by other investigators. Using van der Poel's nomographs and incorporating the effects of voids found in this investigation, the authors developed new nomographs for mix stiffness. They warned that these were only valid in the range of linear behavior of the mix.

Kirk (31) presented results of fatigue tests on specimens 5 cm. X 7 cm. X 35 cm., which were tested in a four-point bending machine. The vertical load varied sinusoidally at the rate of 50 cycles per second. Range of test temperatures from -5°C to 20°C was employed, and failure was defined as the point where the load had fallen to 50% of the starting load. Some of the notable conclusions drawn from the study were that binder content was the most important contributor to fatigue life of asphaltic mixtures, and that air voids had no effect on the fatigue strain.

Significant contributions have come from Pell since 1960. Some of his papers on fatigue of bituminous road

mixes appear in Kolloid Zeitschnift (July, 1960), Proceedings First International Conference on the Structural Design of Asphalt Pavements, and Proceedings Institution of Civil Engineers (Vol. 31, 1965). Later papers include the two cited in this report (5, 32). The testing machine used was of the form of a rotating bending cantilever. It was thought to be advantageous in that the crack propagation was very rapid, and there was no measureable change in stiffness. Specimens had a maximum diameter of $2\frac{1}{2}$ inches at the neck, which increased continuously to $3\frac{1}{2}$ inches near both ends. They were submerged in a coolant liquid. Preliminary work had demonstrated that there was no difference between cooling by liquid or by air, although the latter produced more experimental scatter. Different test temperatures were used, and a constant speed of 1000 rpm. Stiffness was measured by a special machine which applied a constant amplitude deformation to the specimen. The study confirmed that strain was a criterion for fatigue cracking and that stiffness was important in determining fatigue life. Addition of filler in coarse graded mixes resulted in longer fatigue life due to the reduction of voids and an increase in stiffness. It was also noted that there was an optimum asphalt content for maximum fatigue life. Thus, concluding their study, Pell and Taylor (32) observed that the most important factors leading to a

condition of linear mix behavior were low void content, low testing temperature and adequate quantities of filler and binder.

Vallerga et al. (10) investigated the effect of asphalt aging on the fatigue of asphaltic concrete. Their results are significant in that they indicate that stiffness per se is not a determining factor of fatigue life. The test procedure involved aging of asphalt by the Rolling Thin Film Oven method to obtain different consistencies. Unaged asphalt of similar ranges of consistency was also used to manufacture beam specimens of asphaltic mixtures. The specimens were tested in simple beam loading at the rate of 100 applications per minute (with a load duration of 0.1 seconds). It was found that although stiffness increased fatigue life, specimens made with aged asphalt outperformed those made with unaged asphalt for the same range of stiffness. From field data it was found that an increase in the base modulus was significant in increasing fatigue life for both aged and unaged asphalts.

Another paper of interest here is that presented by Schmidt and Santucci (33) in 1969. Their study involved the properties of asphalt as relates to fatigue cracking on a road test project (Zaca-Wigmore). Schmidt suggested an expression which describes the influence of asphalt viscosity

on fatigue resistance of flexible pavements measured under controlled strain. The expression appears below:

$$N_1 = K\, N_o\, (\eta o/\eta_1)\, (1/\varepsilon)^n$$

where K is a constant dependent on the mix, N_o is fatigue life of a reference sample at temperature T_o,

ε is the dynamic strain,

η_o is viscosity of asphalt in the reference sample at T_o

η_1 is viscosity of asphalt at temperature T_1,

N_1 is fatigue life at temperature T_1,

n is a constant varying from 4 to 6.

Thus, the authors claimed, viscosity could be used to predict fatigue cracking of an asphaltic concrete pavement at any reasonable prevailing field temperature. Although the equation suggests that doubling the viscosity results in halving the pavement fatigue life, field observations did not confirm this. In fact data showed that a thousandfold drop in viscosity was required to merely double the pavement life. This was attributed to the fact that the loading conditions were not of controlled-strain type, but lay somewhere between controlled-stress and controlled-strain. As we noted earlier, these two modes of testing are nearly opposed to each other, so that with controlled-stress fatigue life increases as viscosity increases, while for controlled-strain fatigue life decreases with increasing viscosity.

The last paper to be reviewed here was presented by Epps and Monismith (23). In their study the authors investigated the influence of stiffness, air voids, aggregate-gradation and type, and asphalt content on the fatigue life of asphaltic concrete. Significant findings of the study were:

(a) High air voids resulted in shorter fatigue life. The structure of the voids as well as their absolute volume were important because of their effect on load carrying capacity and stress concentration.

(b) Stiffness increased with reduction in air voids. Fatigue life increased with stiffness for a particular stress level. However, they observed that "the change in fatigue life due to variations in air void content cannot be wholly explained by change in stiffness produced by the same air void variation" (23, p. 433).

(c) Aggregate gradation was thought to have an effect on fatigue life for controlled-stress tests.

(d) Aggregate type had an influence on fatigue life.

(e) Average stiffness values of 656,000 psi, 318,000 psi, and 288,000 psi were obtained for asphalt grades of 40-50, 60-70, and 85-100 respectively. Thus, harder asphalts demonstrated higher stiffnesses and longer fatigue lives.

(f) An optimum asphalt content existed for maximum fatigue endurance, all other things being equal. This optimum asphalt content was higher than that required on the basis of stability.

From this review it is evident that the problem of fatigue failure has yet to be resolved. Variability in mix properties and testing procedures result in a wide spectrum of test results and interpretations. Therefore, a number of questions remain unanswered, such as, for example, why asphalt aging should contribute to longer fatigue life for the same range of stiffnesses, or why stiffness and fatigue life do not vary consistently with the factors that are believed to influence them. The results so far show great promise, and it is hoped that fatigue resistance may soon be incorporated as a criterion for the design of flexible pavements. It is evident that laboratory test procedures most beneficial to fatigue testing are those which best simulate field conditions. Among those available today is the deflectometer discussed earlier. This was used in the present investigation.

CHAPTER 3

MIX DESIGN

The mixture used in this study was designed for film thickness according to the Hveem Method of Mix Design (34). Slight departure from the standard procedure was made in sample compaction. This chapter describes the design method.

Materials

The aggregate, which was supplied by the State Highway Department, was taken from Pit Number 75-500 on the Tucson-Nogales highway (U.S. Interstate Highway 19). The contractor for the project was New Pueblo. Three nominal aggregate sizes of 3/4", 3/8" and "sand" were obtained directly from the hot bins at the hot mix plant. These will be referred to hereafter as "coarse," "medium" and "fine" respectively.

Sieve analysis was carried out on each of the aggregates. The fine material had a high percentage of silty-clay and was very dusty. It was thus necessary to conduct a wet-sieve analysis. The results of this analysis are given in Appendix A.

The aggregates were combined to meet the design specification for the highway project (I-19). A percentage combination of 37-19-44 for coarse-medium-fine was obtained. This agreed with that used by the Highway department. The final gradation is given in Table 1 and Figure 1. Table 2 shows two independent results obtained by the Highway department. These are given here for comparison purposes only.

The optimum binder content was estimated by the CKE method, and a Sand Equivalent test was made to estimate the proportion of clayey or deleterious material in the mix. The results of this analysis are summarized in Table 3. Detailed data, including that on the mixture, is also given in Appendix A.

Mixture

An optimum asphalt content of 5.5% by weight of total mix was adopted. It was decided to compact Hveem specimens (4 inches in diameter by 2.5 inches high) at three asphalt contents -- 5.0%, 5.5%, and 6.0% -- for the purpose of characterizing the mix. A vibratory-kneading compactor was used. This compactor was also used for the large specimens. Therefore, description of the molding procedure will be deferred to the next chapter. Evaluation of this apparatus (26) showed that it yields densities comparable to those obtained by other established procedures

(Marshall or Hveem). Hence, it seemed logical to use this compactor, in lieu of Hveem's kneading compactor, in the characterization of the mixture. Hveem stability and cohesiometer values were determined in the manner described in Reference 34. The results are given in Table 4.

Since one objective of this study was to observe the effects of binder content on the fatigue life, deflectometer specimens were made at the three asphalt contents used above. The procedure for this follows.

Table 1

Sieve Analysis of Combined Aggregate

Sieve Number	Percent Passing	Specification Limits*
1"	100	100
3/4"	96	89-100
3/8"	65	61-75
# 4	46	46-56
8	40	39-47
16	33	28-37
30	23	17-26
50	12	10-17
100	5.5	6-12
200	3.4	2-6

*Design Specifications for the I-19-1 (60) Project-Arizona Highway Department.

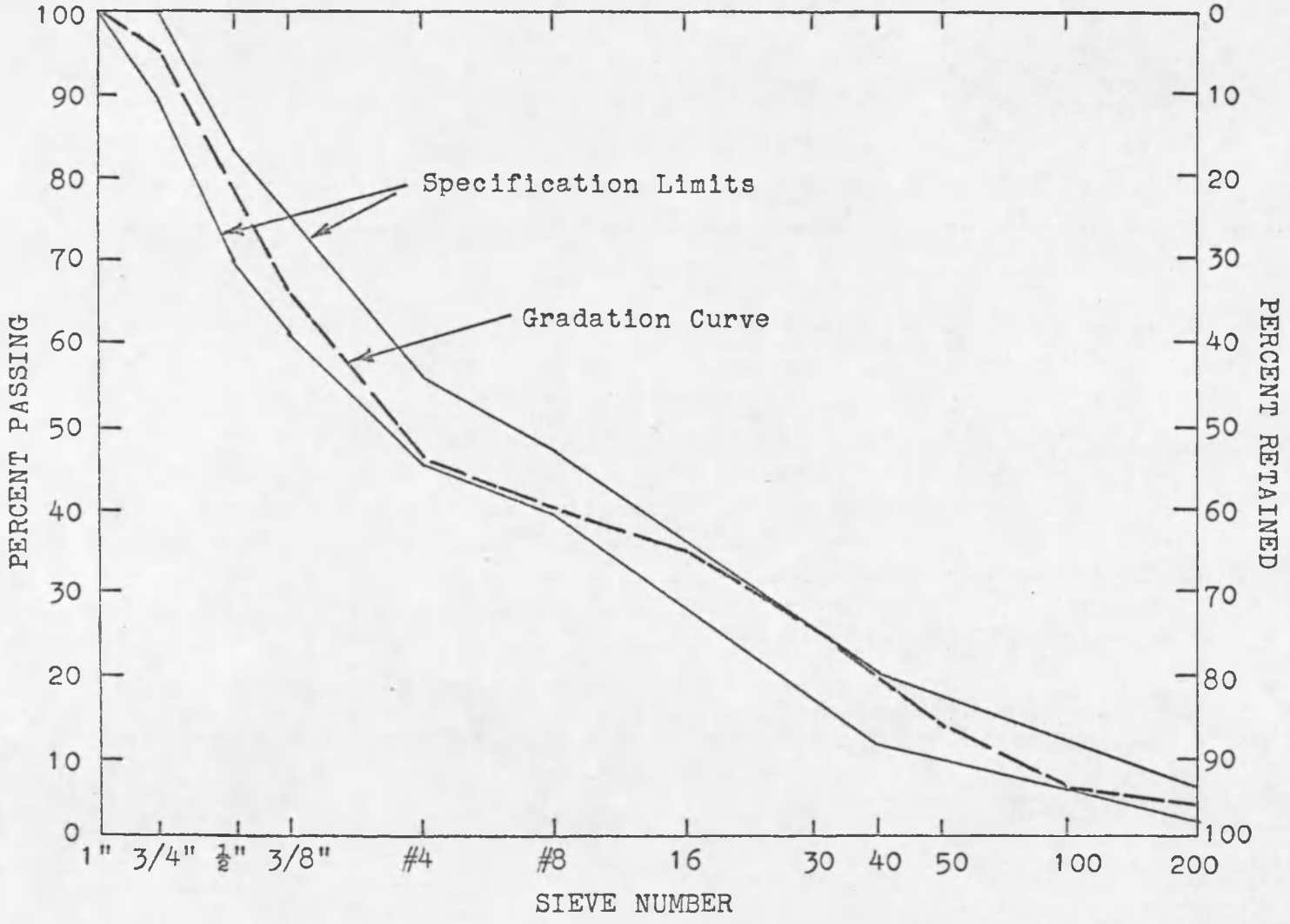


Figure 1. Aggregate Gradation

Table 2

Sieve Analysis of Combined Aggregate

Obtained by Independent Operators

Sieve Number	Percent Passing	
	A*	B
1"	100	100
3/4"	96	95
3/8"	68	65
# 4	51	50
8	43	44
16	32	35
30	21	24
50	12	11
100	7	5
200	5	3

*Courtesy of Arizona Highway Department. Set A was done in the Central Laboratory of the Arizona Highway Department, Phoenix. Set B was obtained by the Highway Department in Tucson. These results are furnished here for comparison purposes.

Table 3

Aggregate Properties

Specific Gravity - Coarse, gm/cc	2.65
- Fine, gm/cc	2.64
- Combined, gm/cc	2.64
CKE - Kerosene Equivalent, %	3.3
- Oil Equivalent, %	2.9
Sand Equivalent, %	47
Surface Area, sq. ft./lb.	25.0
Oil Ratio, % of dry aggregate	3.9
Asphalt Content (60-70 pen.), % of total mix	5.12

Table 4

Design Characteristics of Mixture

Asphalt content, % of total mix	5.0	5.5	6.0
Theoretical Sp. Gr., gm/cc	2.445	2.435	2.414
Specimen Density, gm/cc	2.316	2.371	2.372
Void Content, %	5.2	2.7	1.7
Hveem Stability, %	56	54	42
Cohesiometer Value	225	429	417

CHAPTER 4

SPECIMEN MOLDING PROCEDURE

Preparation of Materials

An estimate of the quantities of different materials required per sample was made based on the densities obtained in the mix design. It was desired to maintain a uniform specimen thickness of 2.0 inches. The diameter was 17.5 inches. Table 5 gives the respective quantities used:

Table 5

Batch Weights, In Grams, Per Specimen

Asphalt Content, % of total	5.0	5.5	6.0
Aggregate - Coarse	6,315	6,280	6,245
- Medium	3,242	3,227	3,212
- Fine	7,505	7,465	7,425
Total weight of Aggregate	17,062	16,972	16,882
Weight of Asphalt	899	988	1,078
Weight per Specimen	17,960	17,960	17,960

Mixing of the mixture was done in a Hobart Food Mixer. Since the quantities involved are large each batch was split into three equal parts and these were mixed

separately. Thus, the different size aggregates were weighed into a large tray (one tray per specimen) and thoroughly mixed my hand. These were then weighed into three equal parts and put in pans for oven heating. The aggregate was put in the oven at 300°F overnight.

Meanwhile, a 60-70 penetration grade asphalt cement was transferred from 5 gallon cans into small cans capable of holding approximately 1,200 gm. of asphalt (the amount required for one specimen). The transfer was done at a temperature of 140°F in order to avoid overheating the binder. A few hours prior to mixing (normally about 4 hours) the asphalt was placed in a small oven and the temperature regulated to 250°F. A thermometer was inserted in the asphalt to monitor the temperature. As soon as it reached the desired consistency mixing was ready to commence.

Mixing

Two minutes before mixing, the mixing bowl was placed in the oven to heat it up to the mixing temperature. The aggregate was then transferred into the bowl and by means of a trowel a depression was made in the center for the binder. A balance capable of weighing up to 15 kgs. was used to tare off the weight of the bowl and its contents. The desired quantity of asphalt was then added--the remaining asphalt was immediately returned to the small oven. The bowl was fixed in the mixing machine and the

operation started. A spatula was occasionally used to clean out material adhering to the sides of the bowl, in order to obtain uniform consistency. Mixing was stopped when the mixture looked uniform. Normally the whole mixing operation took less than 5 minutes.

The mixture was transferred into closed containers (two containers per specimen) and stored in the oven at 300°F until ready for compaction, which normally followed within one hour after mixing. It was desirable to have these two operations over in as short a time as possible in order to eliminate mixture variables due to asphalt hardening.

The mixing room and the compaction room were in separate locations of the building. Ovens were available in the compaction room into which the mixture-containers were transferred. Before compaction started the molds and compaction head were heated using a hot plate. Kerosene was liberally applied on all surfaces which would come into contact with the mixture in order to minimize the risk of adhering. Once the stage was set compaction began.

The Vibratory-Kneading Compactor

A photograph of the compactor is shown in Figure 2. The main features of the machine are, a tilted turntable on which the mold is set, a loading system of counter-rotating eccentric masses, a ballast for dead load, electric motors,

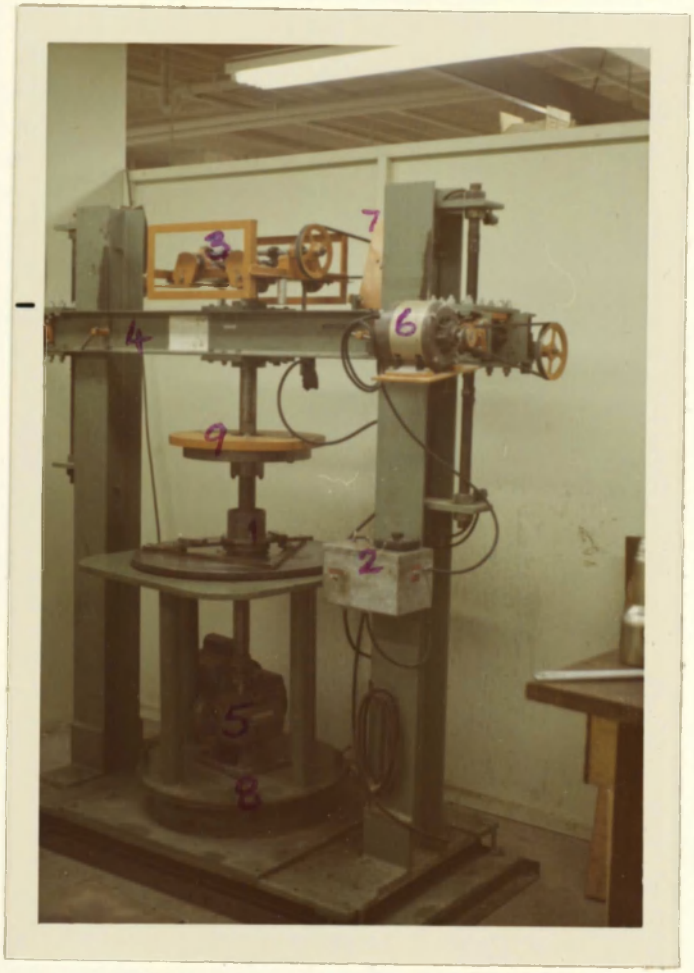


Figure 2. Vibratory-Kneading Compactor

1. 4-inches diameter mold on tilted turntable
2. Electric switches and timer
3. Loading system of counter-rotating eccentric masses
4. Positioning crossbar
5. Electric gear motor for turntable
6. Electric motor for positioning crossbar
7. Electric motor for loading system
8. Tilt control for turntable
9. Ballast for dead load

switches and a compaction head. Two compaction heads were available for the 4-inches and 17½-inches diameter specimens. A detailed description of the apparatus and procedure is given in Reference 9, and is summarized below:

(a) The dead load was 283 lbs. (225 lbs. in the case of 4" diameter specimens).

(b) A dynamic load of 400 lbs. (250 lbs. for the 4" diameter samples) was imparted by the rotation of eccentric masses at a frequency of 1200 revolutions per minute.

(c) A folding-type work-table was attached to the turntable and the heated mold was set on it. A ¼"-steel and ¼"-plywood plates, and a paper disc each 17½ inches in diameter were placed inside the mold. The paper was wetted with kerosene to avoid sticking.

(d) Compaction was done in two layers--the mixture in each container forming a layer. After introducing the hot mixture in the mold a trowel was used to spread it evenly, and a tamping foot was used over the surface. Care was taken to avoid segregation.

(e) The mold was placed on the turntable and the compaction head, ballast, and loading system were lowered into position. A tilt of one

degree was given to the turntable. During the compression of the mixture the whole loading system was resting freely on the mixture.

(f) The timer was set for two minutes and the master switch was set on. During compaction the turntable and mold rotate at 17.5 revolutions per minute. The rotating eccentrics cause a vibratory compaction while the tilt insures a kneading action.

(g) After two minutes the load was removed and the surface lightly scarified before putting on the second layer. The above procedure was repeated.

(h) The tilt was removed and a levelling load (vibratory) was applied for 30 seconds.

(i) The loading system was raised and the mold was removed from the turntable. To extrude the specimen the mold was set on a cylinder about 15 inches in diameter. It was then forced downwards by pulling on it. With proper heating and application of kerosene it was found that the mold came clear very easily.

(j) The specimen was placed on a flat bench and allowed to cool before it was labelled.

Curing

There was no specified curing time for the specimens. However, to limit the risks of damage the specimens were left in position for at least two days before they were moved to another bench for density determination. Afterwards they were transferred into a controlled-temperature room where they stayed for at least four days to achieve the test temperature. In general, specimens were cured for at least ten days.

CHAPTER 5

TESTING PROCEDURE

In this chapter a description of the test equipment and the method of testing is presented. For a better understanding of the development of this procedure the reader is referred to the original work (9).

The two variables controlled during testing were temperature (for all three asphalt contents) and initial support pressure (for the optimum asphalt content). These are shown in Appendix B.

The Deflectometer

The primary components of the deflectometer are, a loading system and a reaction unit. The photograph in Figure 3 shows the equipment fully assembled and ready for testing.

The loading system consists of rotating eccentric and mirror-positioned masses rotating at 740 revolutions per minute. These are driven by a motor which is mounted on the same platform as the masses, and which contributes to the total load on the specimen. A ballast for dead load is also mounted on the shaft. The number of loading is recorded by a counter driven by the shaft rotating the

Figure 3. Deflectometer

1. Reaction Unit Supporting 17½" diameter specimen
2. Loading system of counter-rotating eccentric masses
3. Displacement pump
4. Pressure gauge for reaction unit
5. Specimen
6. Ballast for dead load
7. Electric motor for driving masses

eccentric masses. One counter reading corresponds to 20 revolutions. With this arrangement the apparatus gives a dead load of 175 lbs. and a live load of 150 lbs. These can, however, be varied.

The reaction unit provides support for the specimen. It comprises a cylindrical chamber containing oil and air, and sealed at the top with a thin rubber membrane. The amount of oil and air is controlled to give a standard support pressure to the specimen. To do this the equipment has to be calibrated at each test temperature, and as often afterwards as is deemed necessary. A specimen is clamped onto the oil chamber by means of a steel ring and 16 bolts. The ring has an internal diameter of 14 inches. The clamping force of each bolt is controlled by using identical springs and metal spacer sleeves. The support pressure is recorded on a bourdon pressure gauge mounted on the side of the oil chamber.

To monitor the deflected shape of the specimen gauges are mounted on a rigid carriage (steel bar) which is secured onto the oil chamber. This is referenced so that it is always in the same location.

Calibration of the Deflectometer

A stiff disc $17\frac{1}{2}$" in diameter consisting of a $\frac{1}{4}$" plywood plate sandwiched between two $\frac{1}{4}$" steel plates was used for calibration. This was clamped onto the oil chamber

by means of the steel ring and bolts. The valve connecting the oil chamber and the oil pump was opened. The pump piston was displaced inwards until a pressure of 1.0 psi was registered on the bourdon gauge (the gauge was tapped lightly in the process). The piston was further displaced 2 inches and the pressure noted. With the equipment in calibration a pressure of 2.50 psi was obtained. A further displacement of 2 inches brought the pressure to 4.05 psi. When the apparatus was not in calibration either air was added (or removed) by blowing into the air pipe, or oil was added (or removed) by means of a tee-connection arrangement. It was sometimes necessary to repeat the operation several times (trial-and-error) before calibration was achieved. After calibration the disc was removed and a specimen clamped in position.

Specimen Testing

Both the deflectometer and the test specimens were placed in the temperature-controlled room for at least four days before use. Calibration was done just prior to actual testing. Temperatures of 59°F, 75°F and 90°F were used. One specimen was tested at 41°F but had to be abandoned after eight days. Initial support pressures of 0.0, 1.0, and 2.0 were also used. The following procedure was adopted:

(a) The specimen was secured into position and the bolts were tightened just enough for the sleeves to make contact. During the progress of the test the bolts loosened and had to be tightened again.

(b) A load disc (or loading foot) of 5.0 square inches was used. For the lower temperature it was necessary to use a higher stress and therefore a disc of 3.142 (π) square inches was used.

(c) The loading system was lowered until the load disc barely touched the specimen.

(d) The gauge carriage was fixed in position and all the gauges were set to zero.

(e) The valve connecting the pump to the chamber was opened and the piston was displaced until a pressure of 1.0 psi (or whatever other pressure was used) was registered on the bourdon gauge. The valve was then closed and remained so during the remainder of the test.

(f) The gauges were re-checked to ensure that they were at zero. The counter was also set to zero.

(g) The loading system was lowered onto the specimen and the motor started.

(h) Readings of deflections and pressure were taken at given number of load repetitions. The initial readings were always closely-spaced, timewise. A sample of data record sheet is shown in Appendix C.

(i) The accumulated deflections of the load disc (central deflection) were plotted against the number of repetitions to monitor the condition of the specimen. This plot was linear until failure occurred. At least two more readings were taken beyond the point of failure to ensure a smooth curve. A typical plot is shown in Figure 4.

(j) After failure the specimen was removed and the crack pattern on the lower side was marked out by chalk. This is shown in the photograph in Figure 5.

(k) Radial stresses and dynamic moduli were computed for two points in the loading history of the specimen. One point coincided with the failure stress and another was arbitrarily chosen mid-way between the initial support pressure and the pressure at failure. The support pressure at this point is referred to, later in the report, as P_m, and the failure pressure as P_f.

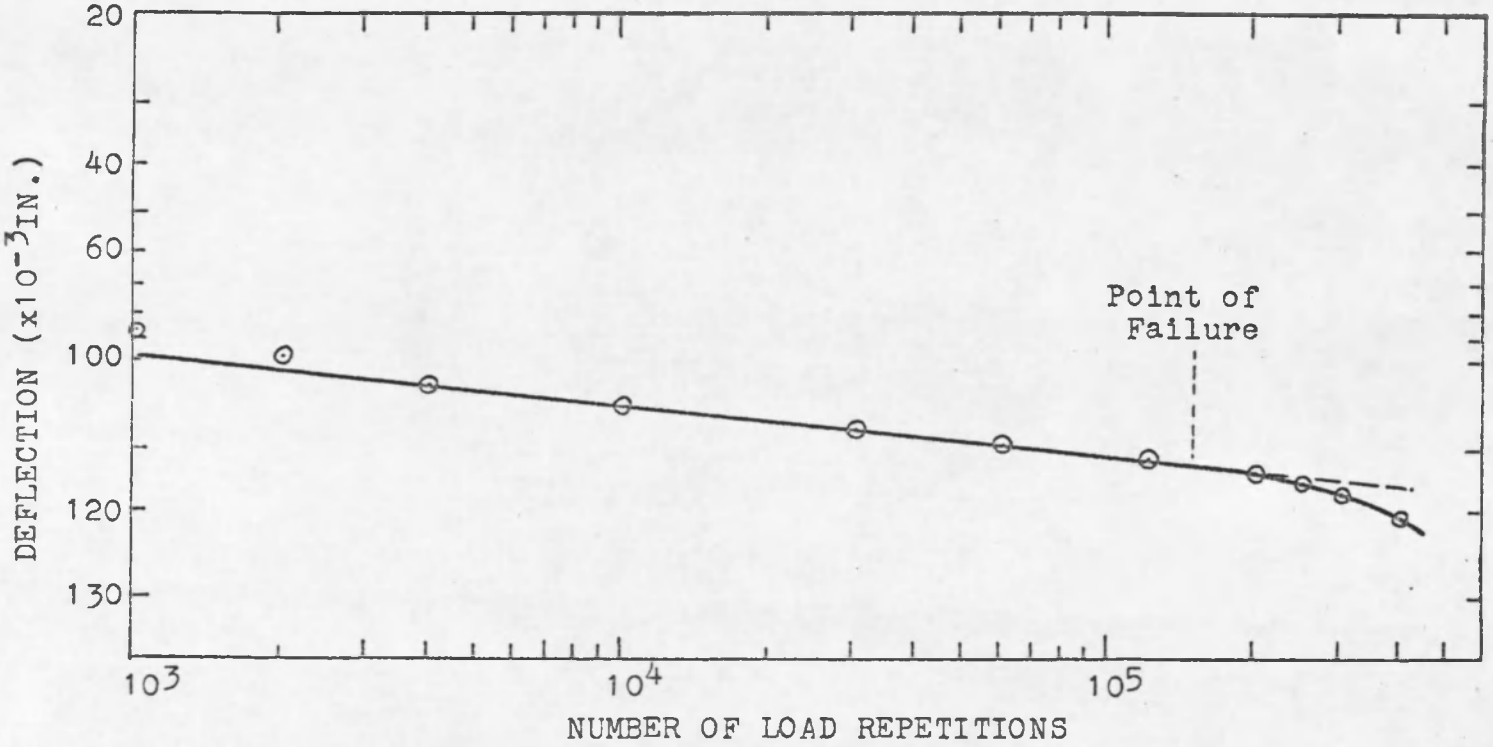


Figure 4. Central Deflection Versus Number of Load Repetitions

Figure 5. Typical Crack Pattern of Failed Specimen.

Stress Computation

Equations used in step (k) above are those given by Jimenez (9) and commonly referred to as Grashof's equations. They were developed from the Bernoulli-Euler theory of bending and modified for dynamic loading. These are given below:

(1) Radial surface stress, S, at the center of a circular disc, fixed about its periphery and supported from below and loaded centrally on top:

$$S = \frac{3(m+1)}{2\pi m t^2} W \left(\ln \frac{r}{r_o}\right) - \frac{3(m+1)\, r^2 P}{8 m t^2} \qquad (a)$$

where m = reciprocal of Poisson's ratio
assumed = 0.2

W = Dead load + load of rotation + load of translation

r = effective specimen radius

r_o = radius of loaded area

P = support pressure at failure

t = specimen thickness

(2) Expression for Modulus of elasticity or the Dynamic Stiffness Modulus:

$$E = \frac{3(m^2-1)}{(4x \text{ rep. defl. at failure } m^2 t^3)} \left[\frac{W^1 r^2}{\pi} - \frac{r^4 P}{4}\right] \qquad (b)$$

Where W^1 = 2 (load of rotation + load of translation) and the other variables are as defined in equation (a).

The equation for load of translation is given as

$$F_t = \frac{Mdw^2}{2}$$

where M = Mass = Total Dead Load

d = repeated deflection

w = angular velocity of the eccentric masses

= 77.5 radians per second

When the values of load, r, r_o and m are substituted in equations (a) and (b), the following equations are obtained.

For a 5.0 square inches load disc:

$$S = \frac{1}{t^2}\left[(319 + 1.331d) - 22.05p\right] \qquad (c)$$

For a π square inches load disc:

$$S = \frac{1}{t^2}\left[(362 + 1.511d) - 22.05p\right] \qquad (d)$$

$$E = \frac{10^3}{8t^3}(3370 + 30.45p - 432p) \qquad (e)$$

where t = specimen thickness

p = support pressure at point of determination

d = repeated deflection at point of determination

and δ = repeated deflection at failure

If the mixture characteristics are determined at failure, then d = δ.

Equations (c), (d) and (e) were used in finding the radial stresses and dynamic moduli of the specimens. The results are discussed in the next chapter. The number of load repetitions to failure were evaluated, for each test, from the running-plot of the deflection-load repetition curve. The point of failure was taken at the tangency point shown in Figure 4. Data obtained using the small load disc was then reduced to obtain the number of repetitions for failure corresponding to a loading area of 5.0 square inches, by calculating the new radial stresses, and assuming the validity of the relationship between stress and load repetitions:

$$S = I_o N^{-b} \quad , \text{ with } b = \tfrac{1}{4}$$

Further reduction was made of the load repetitions to failure at a constant stress-level of 80 psi.

CHAPTER 6

TEST RESULTS AND DISCUSSION

The results of the deflectometer tests are given in Tables 9 to 14. They are also represented graphically in Figures 6 to 14. In this chapter we shall discuss generally the factors which influence the test results, and then examine specifically the effects of temperature, asphalt content, and initial support pressure on stiffness and fatigue life.

The behavior of many engineering materials is a function of their past history--whether they have been subjected to stresses or environmental conditions which may weaken their strength. With asphaltic concrete specimens, care in handling is very important, especially immediately after molding since the material is semi-fluid. Careless handling can therefore weaken the specimen and yield bad results. Although every care was exercised in this study, it should be noted that disturbance to the sample especially around the edges was inevitable during the removal from the mold. As we mentioned earlier, specimens were left undisturbed for at least two days before they were transferred. It is not expected that any damage resulted from these transfers.

The initial curing was at room temperature (about 70°F). The samples were then transferred to the test temperature. This would undoubtedly set up some stresses in the hardened mixture. Whether this has any considerable effect on the subsequent behavior of the specimen is unknown.

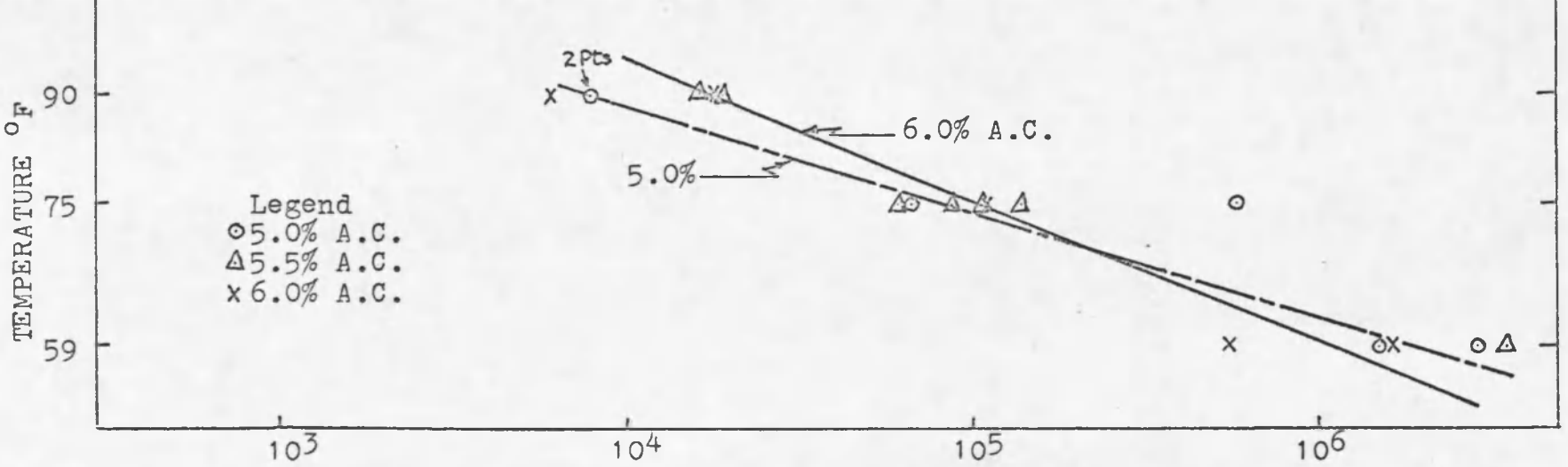
An important aspect of the mix believed to affect the results is aggregate gradation. As will be seen from Figure 1 the aggregate is gap-graded and has a high percentage of coarse aggregate. This results in a rough surface texture. Although this is not unusual for a highway, in a laboratory test it may lead to erratic results due to the uneven distribution of stresses on the load disc. Thus some portions of the loaded area may be overstressed, and failure will be localized.

The effect of the coarse gradation is also manifested in the air voids obtained. Average voids for 5%, 5.5% and 6% asphalt contents are 6.3, 5.0 and 4.2%, with a range of values from 5.0 to 8.1% for 6 samples, 2.7 to 7.4% for 18 samples, and 1.9 to 5.5% for 6 samples, respectively. These are higher than those obtained from the 4" samples. However, they show the expected influence of asphalt on void contents. The large scatter is attributed to the nature of the mix. Although care was taken to avoid segregation, it was noted, on coring one of the

specimens, that some pockets of voids were visible. This was a local phenomenon only and may have affected different specimens differently. For although specimens could have equal air voids, the location or distribution of the voids as well as shape and size have been known to influence strength performance considerably (23).

Temperature Effects

The influence of temperature on the test results is examined for two test conditions: (a) Variable asphalt content and uniform initial support of 1.0 psi, and (b) constant asphalt content of 5.5% and variable initial support. Figure 6 shows the effect of temperature on fatigue life on a log-log scale. It is seen that the effect is essentially linear, and that fatigue life increases as the temperature decreases. The lower asphalt content yields a flatter curve indicating that the samples are less susceptible to temperature variability. This is probably due to the film thickness, since at the higher asphalt content the mixture has a thicker film than it is designed for. Therefore its behavior approaches that of the binder which is thermoplastic. The effect of initial support pressure is not clearly defined because of the scatter in the results. However, it appears that medium support of 1.0 psi gives a better performance for the same test temperature. It was noted that the effect of zero initial support was to induce



6 (a). 1.0 psi Initial Support Pressure

6 (b). 5.5% Asphalt Content

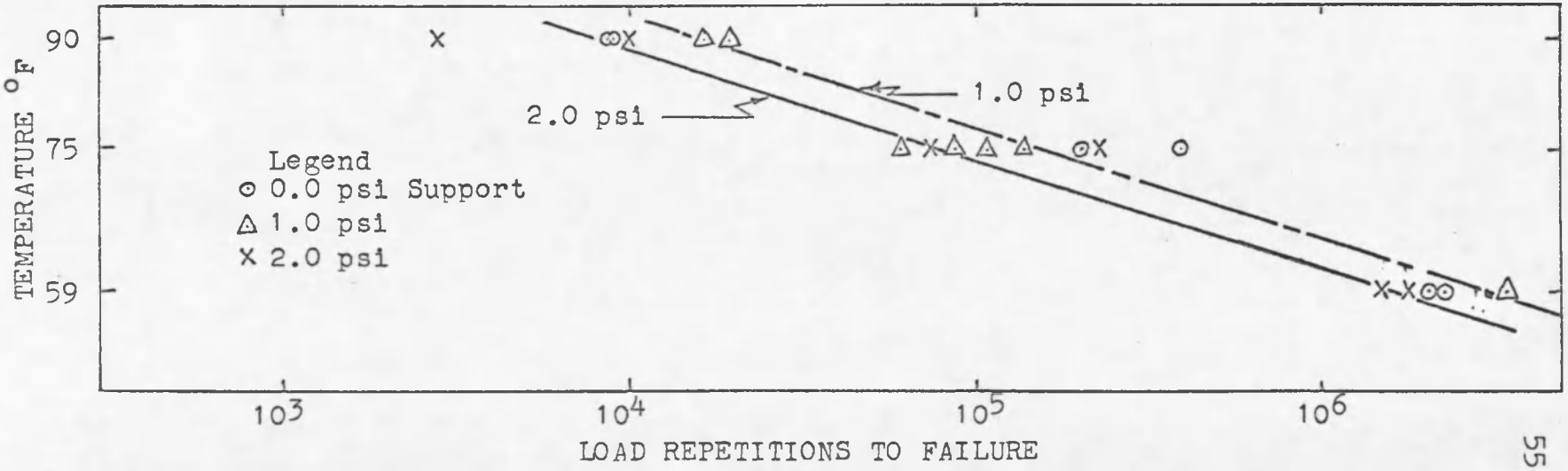


Figure 6. Effect of Temperature on Fatigue Life

a large initial deflection in the specimen, but otherwise the fatigue life was comparable to that given by the higher support. These will be discussed more in a later section of this chapter.

Figures 7a and 7b show the influence of temperature on the dynamic stiffness modulus. A nonlinear relationship is indicated but this also varies with asphalt content and support pressure. The modulus increases as the temperature decreases. This was to be expected since the asphalt viscosity, in poises, increases about two orders of magnitude for a temperature drop from 90°F to 59°F. But the precise effect this change in viscosity has on a mixture stiffness is still a matter of conjecture. For example, it is indicated that for low asphalt contents there is an optimum temperature below which the modulus will decrease. This optimum is even more pronounced for a zero initial support (see Figure 7b). Similar results were obtained by the WASHO road test, where an optimum temperature existed for a maximum modulus of rupture. This is shown in Reference 11.

Figures 8a and 8b show that the cumulative central deflection varies with temperature, and that the optimum temperature mentioned above also results in minimum deflection. The magnitude of the deflection is greatly influenced by the initial support pressure. It was noted that for zero support most of the deflection was instantaneous.

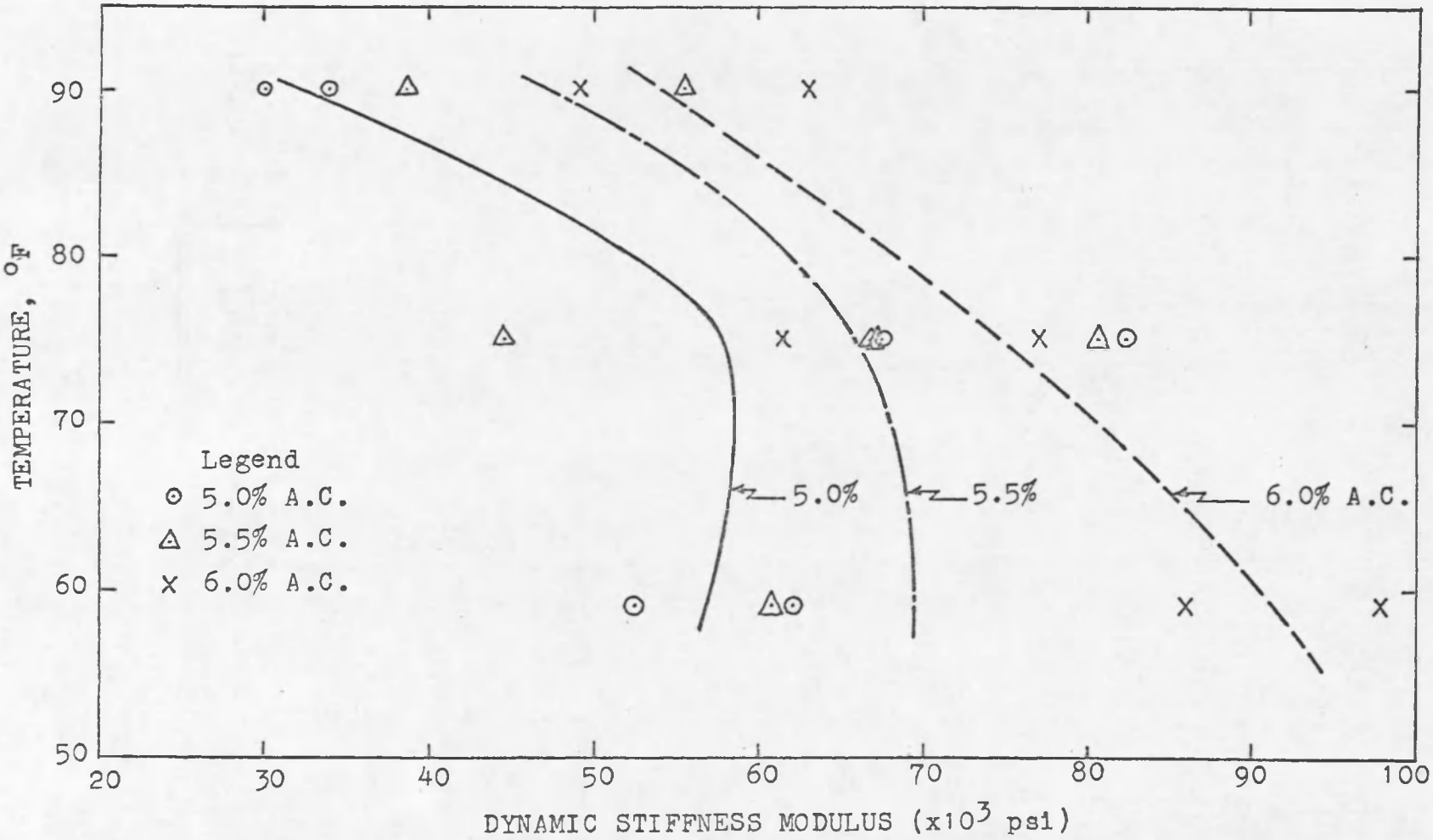


Figure 7. Effect of Temperature on Stiffness
a. Initial Support Pressure of 1.0 psi.

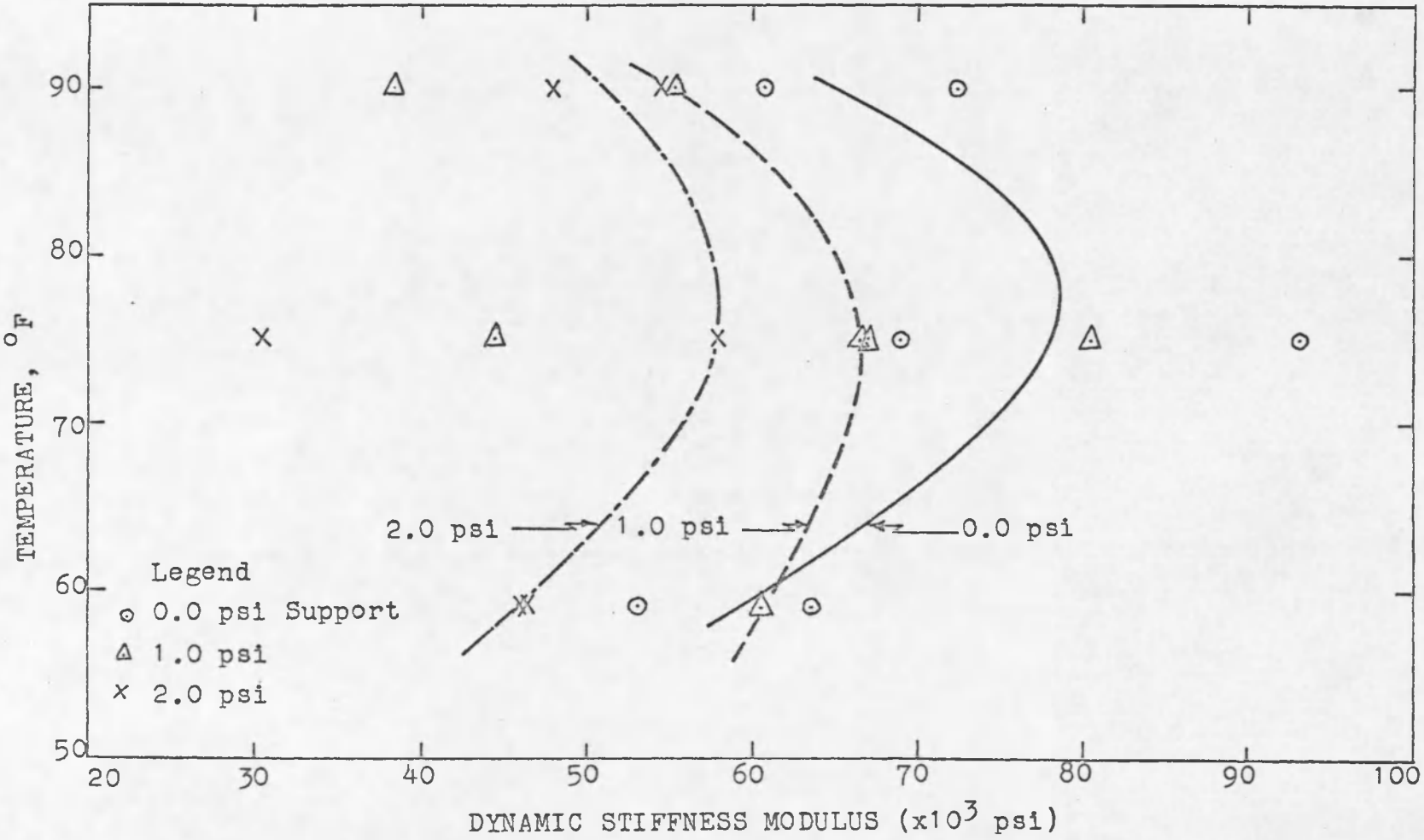


Figure 7, Continued.
b. Asphalt Content of 5.5%.

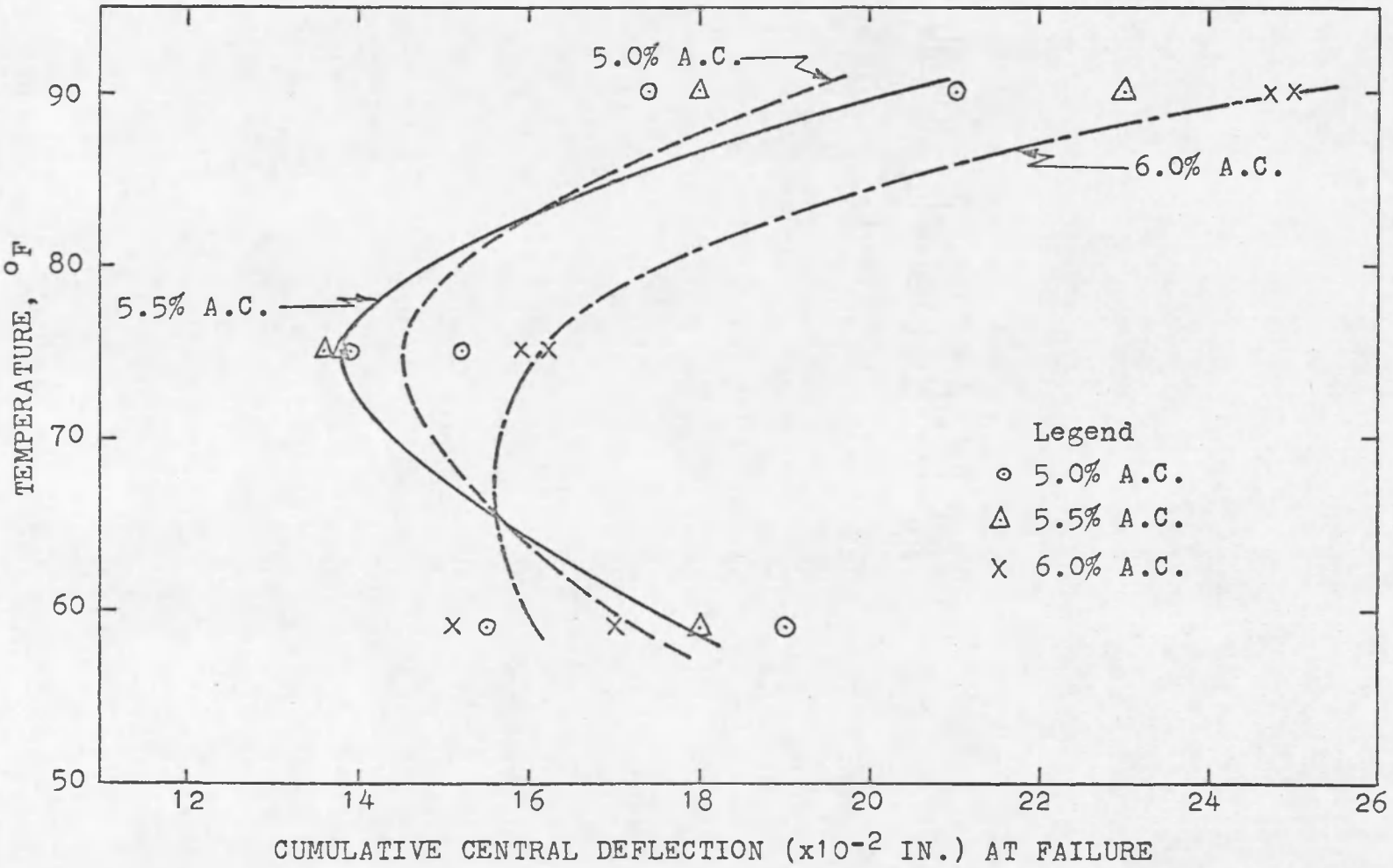


Figure 8. Effect of Temperature on Total Deflection
a. Initial Support of 1.0 psi.

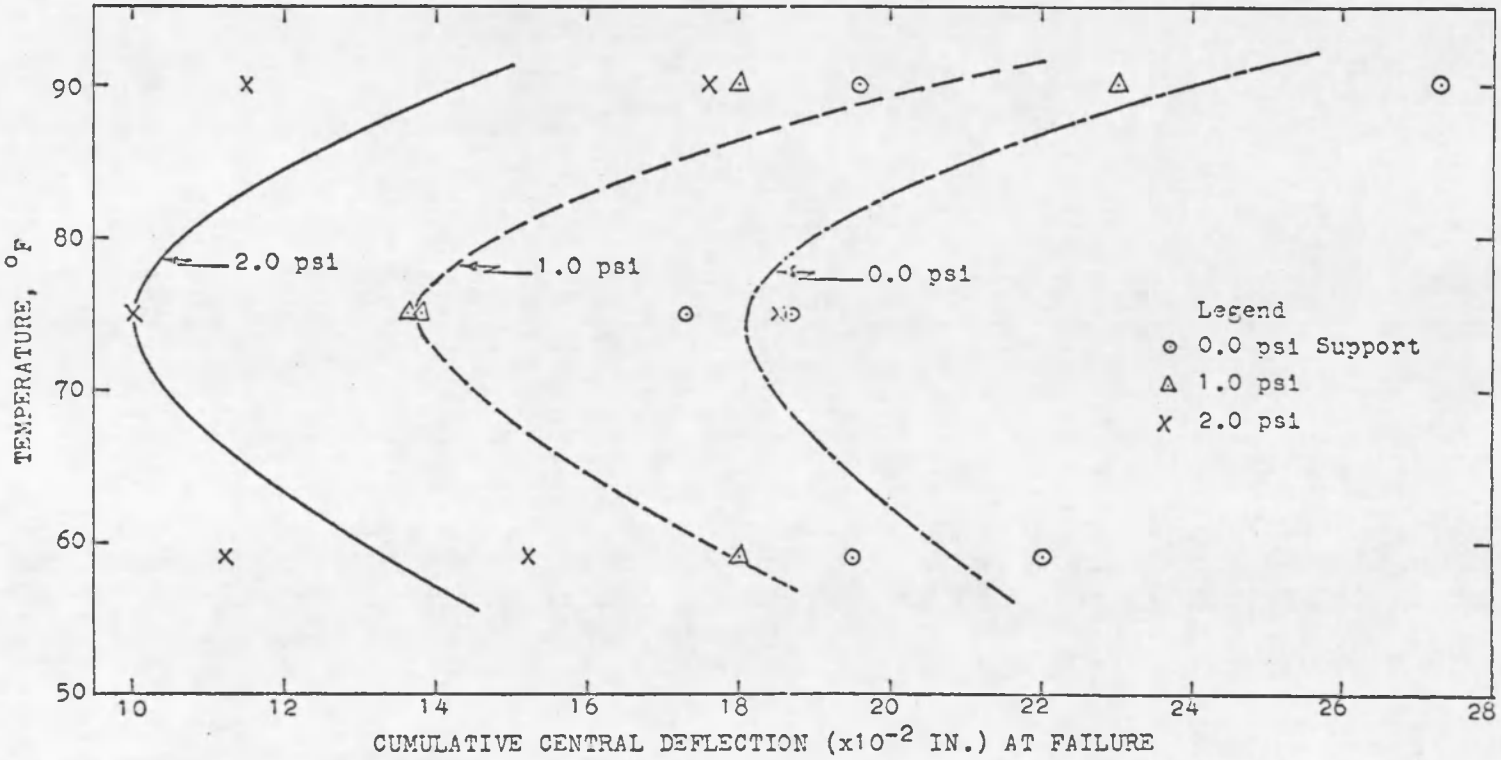


Figure 8, Continued.
b. Effect of Temperature on Total Deflection for Asphalt Content of 5.5%.

However, as we shall see later, this did not seem to impair the fatigue performance of the specimen.

Effect of Asphalt Content

Figure 9 shows that there is an optimum asphalt content associated with maximum fatigue life, for each test temperature. For this mixture it is about 5.6%. It will be recalled that the estimated binder according to the CKE test was 5.1%. This would suggest that the desirable binder content for fatigue life is higher than that obtained by the Hveem Method on the basis of stability.

In Reference 9 Jimenez also reported an optimum asphalt content for maximum fatigue life. He explained that this may be due to excessive film thickness (in the case of the higher asphalt contents) resulting in reduced frictional resistance. It was also suggested that the crack pattern on mixture surfaces which are deficient in asphalt is different from that of mixtures rich in asphalt. In this study cracks in specimens containing 6.0% asphalt were not clearly defined and so no distinction could be made. A typical crack pattern observed in a 5.0% asphalt specimen was shown in Figure 5. For comparison, Figure 11 is a photograph of a cracked section of a street in Tucson. The crack patterns are similar, indicating that "alligatoring" is a characteristic fatigue failure of asphaltic pavements which are deficient in binder content. This may seem like

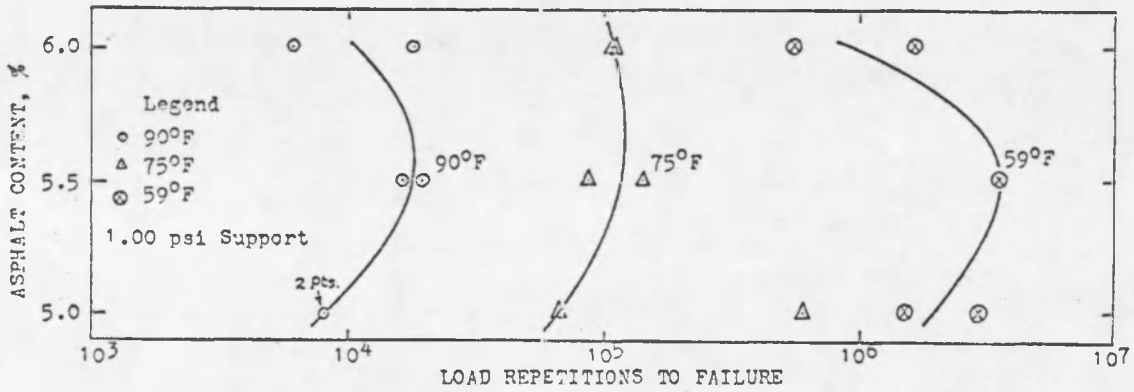


Figure 9. Effect of Asphalt Content on Fatigue

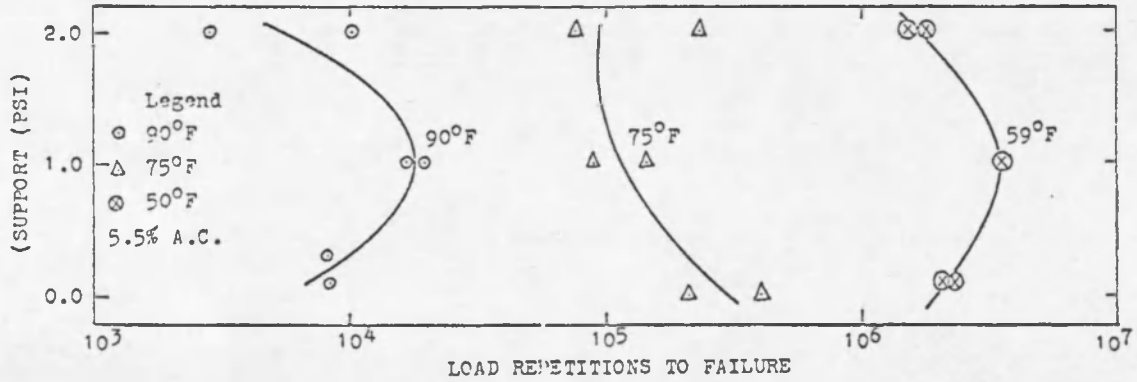


Figure 10. Initial Support Pressure Versus Load Repetitions to Failure.

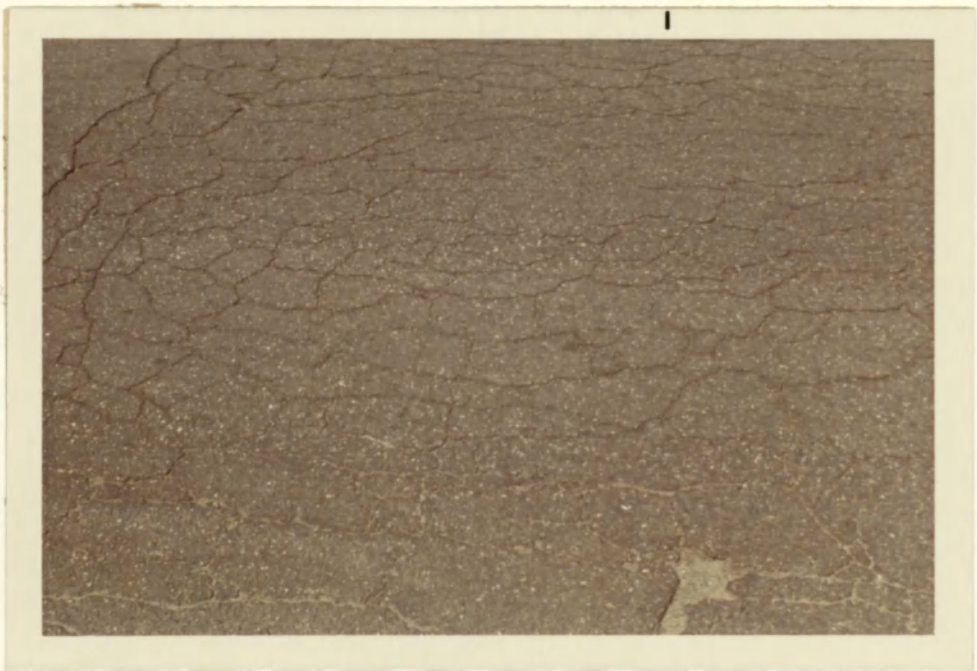


Figure 11. "Alligator" Cracking of an Asphaltic Concrete Pavement.

stating the obvious, since pavements are designed for stability or film thickness and hence a lower asphalt content than is required for fatigue. Yet many pavements would last longer if this point was given serious consideration by designers.

Effect of Initial Support

Observed results on the influence of initial support pressure on fatigue life show some inconsistencies which cannot be simply explained (Figure 10). For a temperature of 75°F, which was noted to be approximately optimum for maximum stiffness and minimum central deflection, the lower the initial support the higher the fatigue life. However, for temperatures of 59°F and 90°F, an initial support of 1.00 psi is optimum for best fatigue resistance.

These findings are not in accord with what has been reported previously (9). It was shown that a linear increase in fatigue life with support pressure existed. However, it should be pointed out that the two mixtures vary considerably in composition and physical properties. As Pell and Taylor (32) noted, mixtures with a fine gradation, low voids, and adequate filler and binder have a linear behavior under load stresses. This mixture, to the contrary, was coarse-grained and gap-graded.

Nevertheless, an optimum initial support, as indicated, may be significant for flexible pavements on elastic

foundations. Hveem (4) postulated that the "principal destructive force" in fatigue cracking was the "energy stored in the subgrade by each passing wheel." Such energy or rebound would certainly be higher for the higher support. But for the low (zero) support, the larger deflections would tend to reduce the fatigue resistance, except at the optimum temperature where stiffness and flexibility combine to give better performance.

Dynamic Stiffness Modulus

In an earlier section the effect of temperature on the dynamic stiffness was discussed. It has also been noted that stiffness is affected by a number of variables such as aggregate type and gradation, asphalt consistency, void content and stress level. The last two variables will be discussed here.

Figures 12a, b and c show the variation of air voids with stiffness for test temperatures of 59°F, 75°F and 90°F, respectively. Several other variables are involved, as will be seen. However, a general pattern is evident irrespective of asphalt content or initial support pressure: the stiffness increases as the void content decreases. The effect of temperature is to produce a family of curves which are parallel. These are shown in Figure 12d. From this figure we see that for the same void content the stiffness is very much a function of temperature.

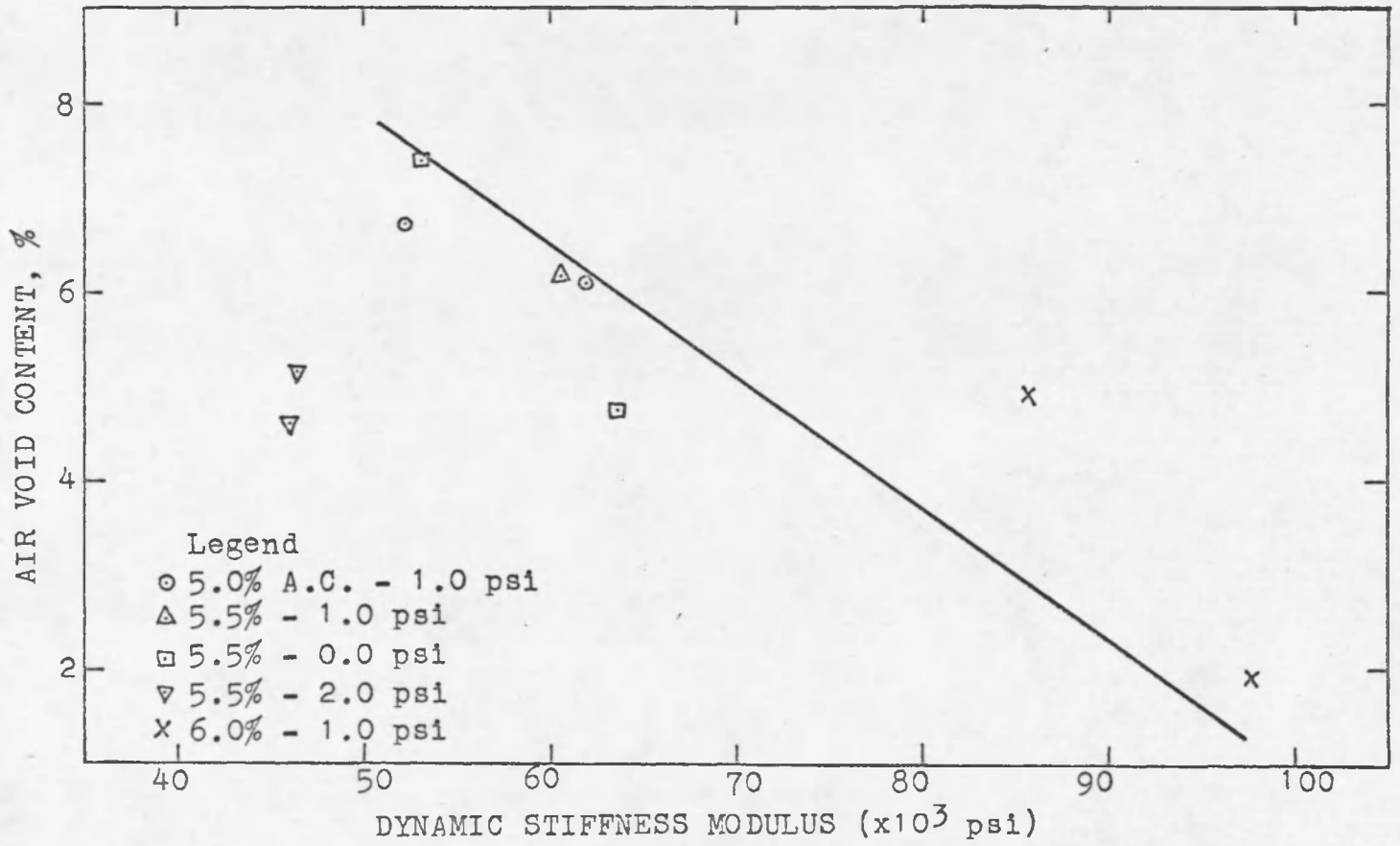


Figure 12. Effect of Voids on Stiffness at Various Temperatures
a. At 59°F

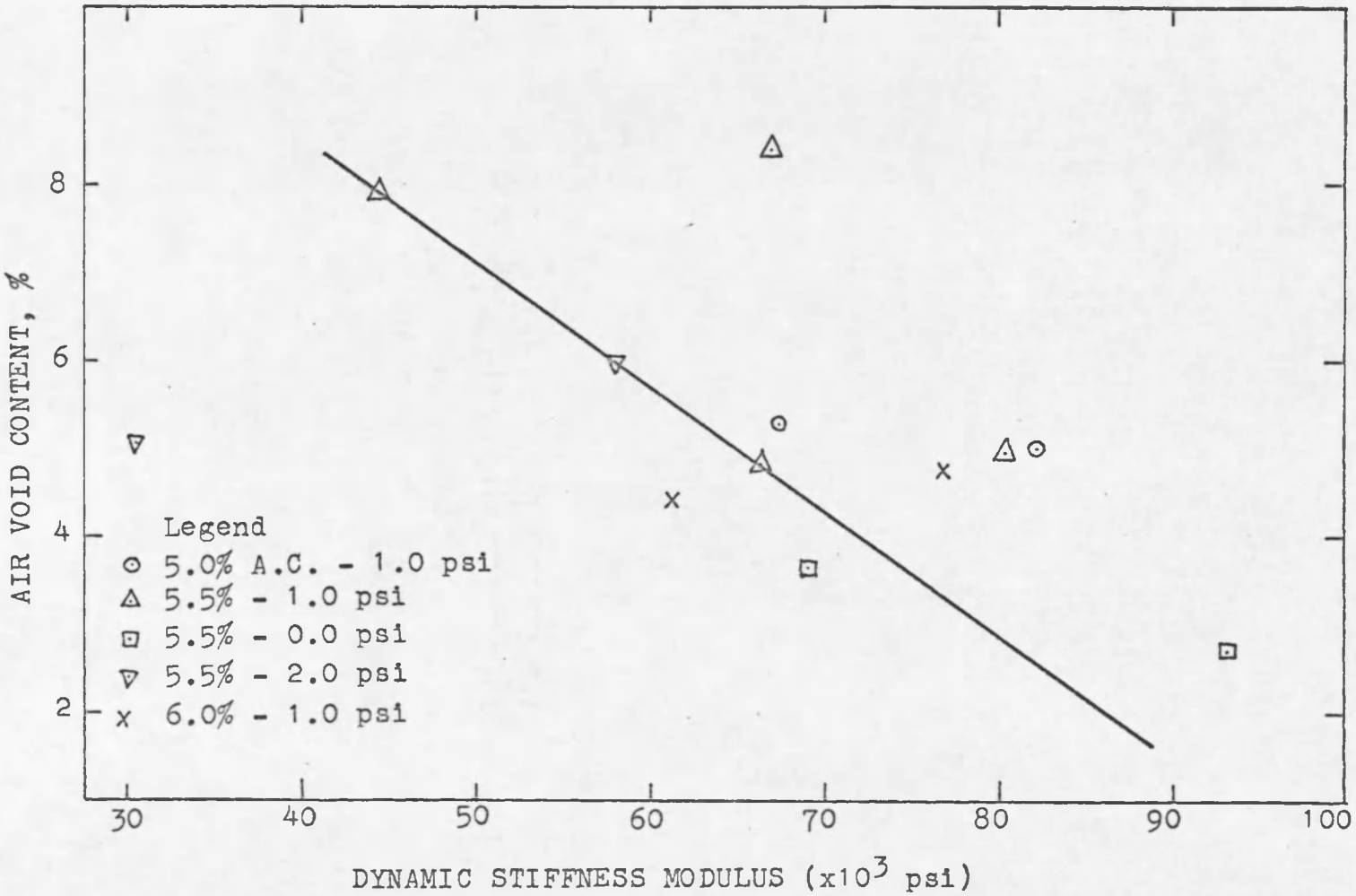


Figure 12, Continued.
b. At 75°F

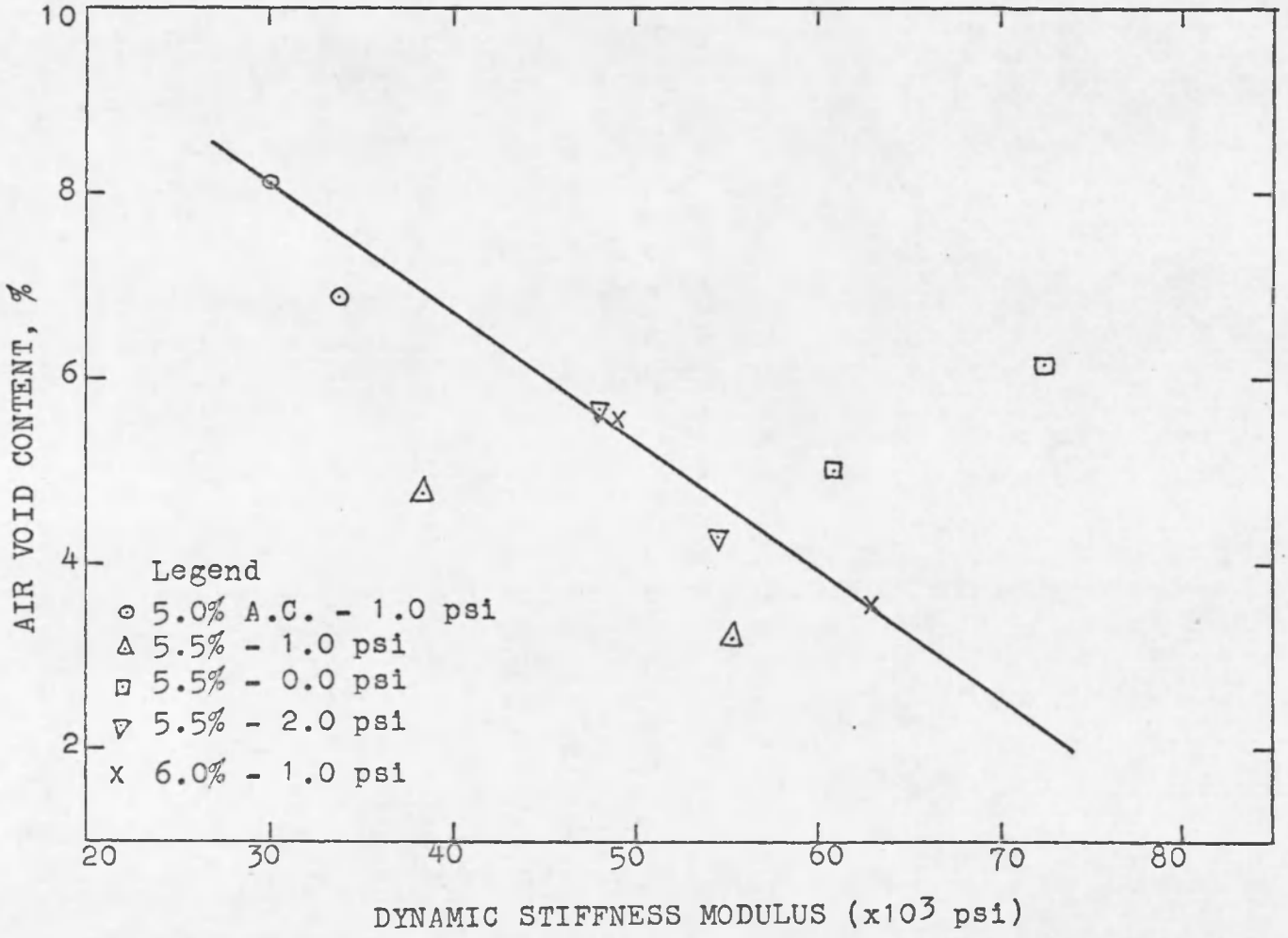


Figure 12, Continued.
c. At 90°F

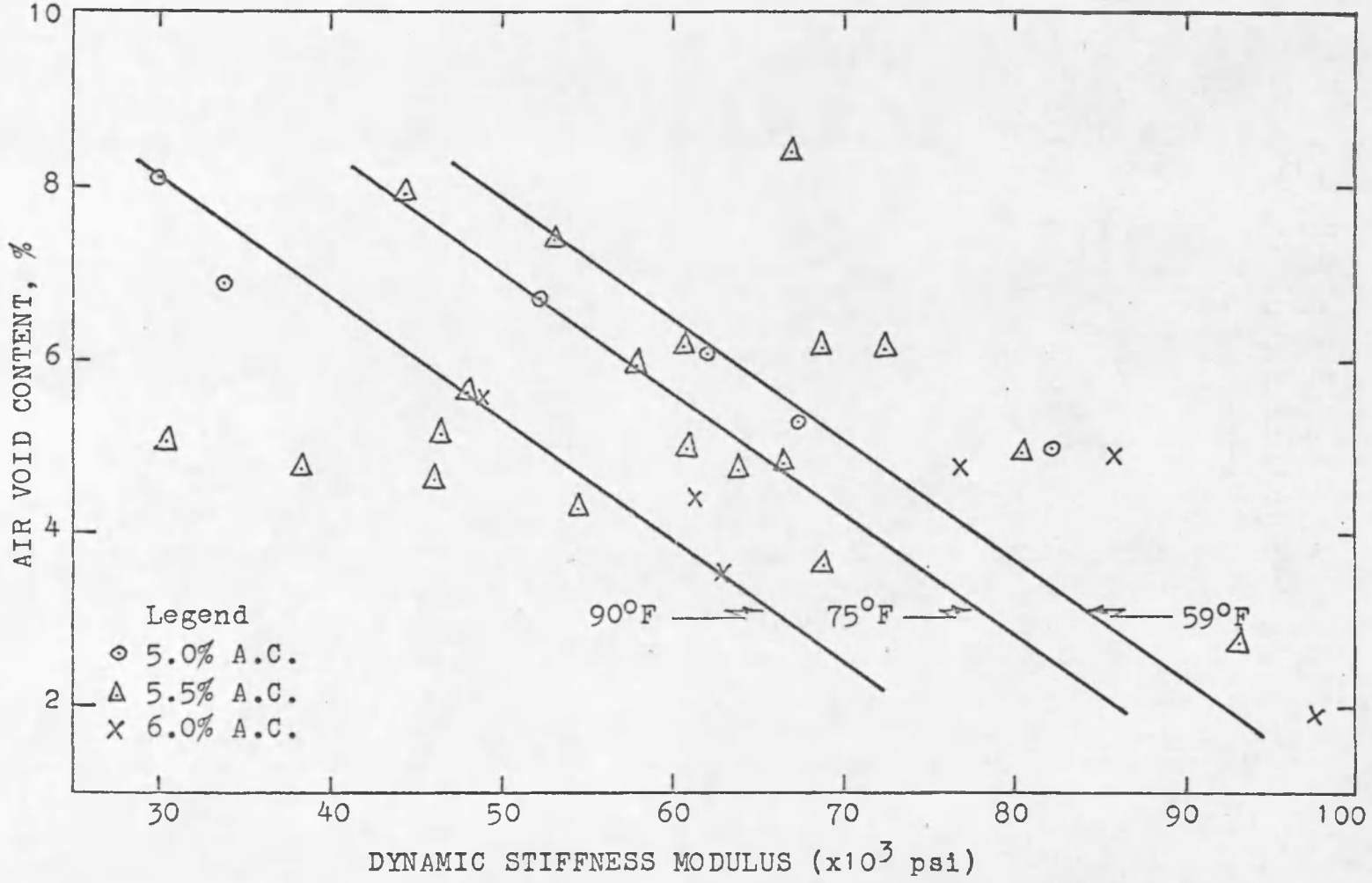


Figure 12, Continued.
d. At any temperature.

Although the deflectometer is essentially a controlled stress device, variations in stress occur due to variations in the translation force. The variability of the radial stresses with dynamic stiffness is shown in Figure 13. The effects of different temperatures, asphalt contents and initial support pressures were not apparent, and so no attempt has been made to isolate all these variables. It is seen from this that the radial stresses increase as the stiffness increases. But since the fatigue resistance is dependent on the stress level (8, 9) the contribution made by stiffness to stresses may be detrimental to the fatigue life of the mixture. However, a number of other factors which contribute to the higher stiffness also contribute to longer fatigue life.

The variability of stiffness with load repetitions to failure is shown in Figure 14. This is at a stress-level of 80 psi. Again no attempt has been made to isolate the many variables, although it would be desirable to do so. This is because of the very limited number of samples available (even for replicate samples there is variability in void content which has to be accounted for). A general trend is, however, indicated that an increase in stiffness results in longer fatigue life.

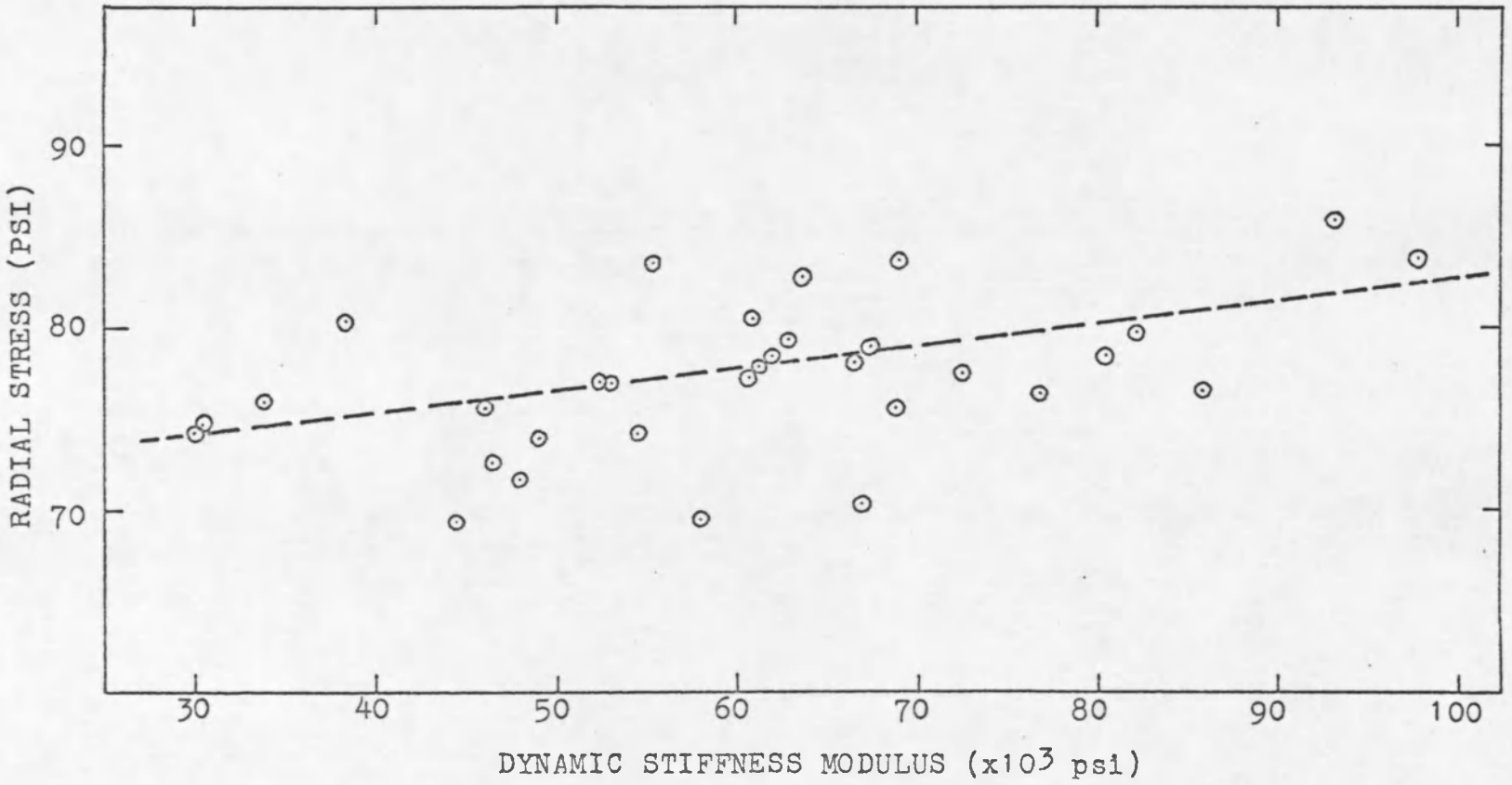


Figure 13. Variation of Radial Surface Stresses with Stiffness.

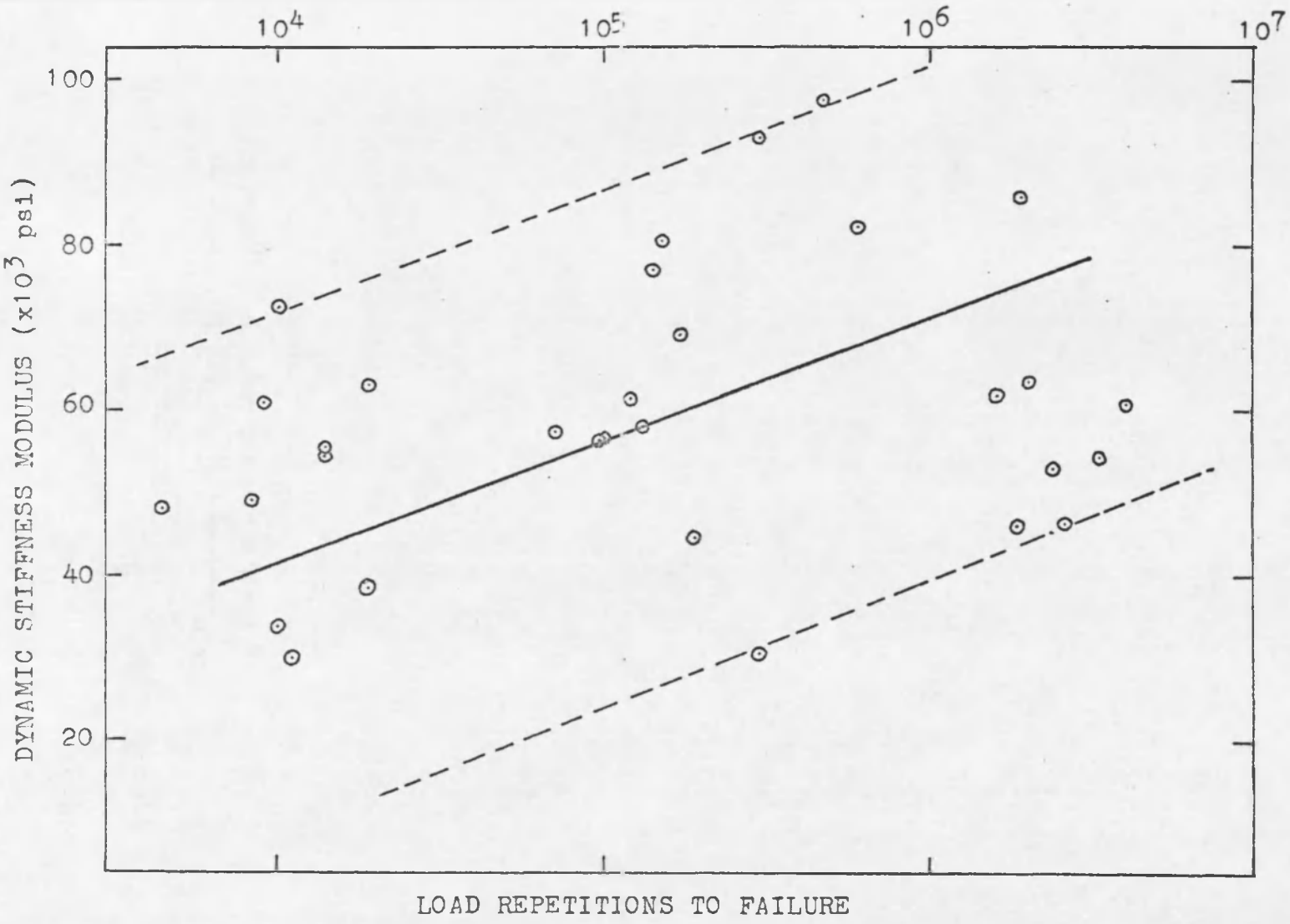


Figure 14. Variation of Stiffness with Load Repetitions to Failure.

CHAPTER 7

SUMMARY AND CONCLUSIONS

This thesis has attempted to give a broad outline of the influence of mixture stiffness on fatigue resistance. Numerous factors affecting stiffness were discussed. Because of the limited number of samples tested, it is not possible to give more than a generalized picture of the observed relationships. As Deacon and Monismith (20, p. 28) pointed out, fatigue life is a "stochastic rather than deterministic phenomenon," and hence techniques of analysis should employ statistical methods.

The following is a summary of the findings reported in this study:

(1) Temperature has a profound effect on fatigue life. The fatigue life increases by about one order of magnitude for a temperature drop from 90° to 75° and from 75° to 59° (see Figure 6). It was noted that for the same range of temperatures, asphalt viscosity increases by one order of magnitude. However, these variations were not reflected in the dynamic stiffness moduli.

(2) The influence of temperature on stiffness is non-linear and varies with asphalt content and initial support pressure. For low asphalt contents there is an optimum temperature below which the modulus will decrease. But for asphalt contents greater than optimum the stiffness increases with decreasing temperature (Figure 7a).

(3) There is an optimum temperature (about 75°F) for which the central cumulative deflection at failure is a minimum. This temperature coincides with that required for (2) above.

(4) There is an optimum asphalt content of 5.6% for best fatigue resistance. This was 0.5% higher than that estimated by the Hveem design method.

(5) For the optimum temperature reported in (2) and (3) above, low initial support pressure resulted in better fatigue resistance. This effect was observed only for an asphalt content of 5.5% (slightly lower than the optimum). It would be desirable to investigate this effect with other asphalt contents.

(6) An initial support pressure of 1.00 psi gave best performance for temperatures of 59°F and

90°F. Again this was for an asphalt content of 5.5%. This suggests that large support pressures may be detrimental to fatigue life because of the elastic rebound.

(7) The dynamic stiffness increases with a reduction in air voids. The effect of temperature is to produce a family of parallel curves on the Voids-Stiffness plot.

(8) Although the deflectometer is essentially a controlled stress type of device, variations in radial stresses occur, due to translation of the load. The stresses increase with increasing stiffness.

(9) In general, the fatigue life increases with the dynamic stiffness modulus.

(10) The behavior of the mixture tested was fairly erratic. This tends to confirm earlier reports that coarse graded mixtures lend themselves to non-linear behavior. The radial stresses and stiffnesses calculated at two points in the loading history of each specimen (Tables 12 and 13), indicate a considerable change of these properties during the test, whereas earlier reports (9, 12, 26) had shown these to be fairly constant. Hence,

gradation may have a large influence on the behavior and fatigue resistance of asphaltic concrete mixtures.

CHAPTER 8

SUGGESTED FURTHER RESEARCH

The study reported here is by no means conclusive. The number of samples for each variable was not large enough to allow a statistical analysis. It would be desirable to continue this study in order to be able to establish a general relationship for each of the factors affecting stiffness. In particular, the effects of initial support on fatigue life for a range of asphalt contents, should be investigated.

Another avenue is that dealing with aggregate gradation. As this and other studies have indicated the gradation is important, especially as it influences the amount of binder that can be incorporated in the mix. Gap-graded mixtures and binders of high viscosity have been used with great success in Europe. It would be desirable to evaluate the deflectometer in terms of these.

APPENDIX A

MIX DESIGN

Table 6

Aggregate Combination

Passing	Coarse		Medium		Fine		Combined
Size	Total %	x0.37	Total %	x0.19	Total %	x0.44	Analysis
1"	100.0	37.0	100.0	19.0	100.0	44.0	100.0
3/4"	89.2	33.0	99.6	18.9	100.0	44.0	95.9
3/8"	12.6	4.7	86.5	16.5	100.0	44.0	65.1
# 4	0.3	0.1	13.8	2.6	98.6	43.4	46.1
8	0.1	0.04	0.6	0.1	90.4	39.8	40.0
16	0.1	0.04	0.5	0.1	74.7	32.8	32.9
30	-	-	0.5	0.1	52.6	23.2	23.3
50	-	-	0.4	0.1	26.5	11.7	11.8
100	-	-	-	-	12.5	5.5	5.5
200	-	-	-	-	7.6	3.4	3.4

Table 7

Aggregate Properties

(a) Specific Gravity

Size Range	3/4"-#4	3/4"-#4	Minus #4	Minus #4
Identification	Coarse	Coarse	Fine	Fine
Wt. in Air, gm	1119.0	1167.0	955.0	1009.0
Pyc. + Water, gm	2703.0	2703.0	2703.0	2703.0
Sum, gm	3822.0	3870.0	3658.0	3712.0
Pyc. + Sample + Water	3400.8	3427.7	3297.0	3329.8
Vol. of Sample, cc	421.2	442.3	361.0	382.2
Apparent Sp. Gr. gm/cc	2.658	2.640	2.645	2.641

(b) C.K.E.

Size Range	-#4	-#4	3/8"-#4	3/8"-#4
Identification	I	II	I	II
Sample + Cup, gm.	317.3	318.7	180.0	180.0
Wt. of Cup, gm.	217.3	218.7	80.0	80.0
Wt. of Sample, gm.	100.0	100.0	100.0	100.0
Wet Sample + Cup	320.6	322.1	183.0	182.8
Wt. of Kerosene/Oil	3.3	3.4	3.0	2.8
Absorption, %	3.3	3.4	3.0	2.8

Table 7 (continued)

(c) Sand Equivalent

Identification	2-1	3-1	2-2	3-2
Sand level, in.	3.82	3.80	4.05	3.75
Clay level, in.	8.35	8.32	8.10	8.00
S.E. %	45.7	45.7	50.0	47.0

Calculations

Specific gravity of plus #4 size (coarse) = 2.65

" " " minus #4 " (fine) = 2.64

$$\text{Combined S.G.} = \frac{100}{\frac{\%\text{Finer}}{\text{Fine S.G.}} + \frac{\%\text{ Retained}}{\text{Coarse S.G.}}}$$

From gradation curve % Finer than #4 = 45.3

% Retained = 54.7

$$\text{SG (combined)} = \frac{100}{\frac{45.3}{2.64} + \frac{54.7}{2.65}}$$

$$= \frac{100}{37.82}$$

$$= 2.642$$

Surface Area (S.A.--sq. ft./lb.)

Sieve Size	% Passing	S.A. Factor	Surface Area
Plus # 4	-	-	2.0
# 4	45	2	0.9
8	40	4	1.6
16	34	8	2.7
30	26	14	3.6
50	14	30	4.2
100	06	60	3.6
# 200	04	160	6.4
		Total =	25.0

Surface Constants:

CKE:

Measured CKE = 3.35%

Corrected value = $3.35 \times \frac{2.64}{2.65} = 3.34$

From Asphalt Institute Mix Design Handbook (34, Fig. V-2):

CKE = 3.34

S.A. = 25

% Finer # 4 - 45.3% ——— $K_f = 1.05$

Oil

Oil retained = 2.9%

Corrected oil retained = $2.9 \times \frac{2.65}{2.65}$

= 2.9%

From Fig. V-3 (34) $K_c = 1.3$

From Fig. V-4 (34)

$(K_c - K_f) = 0.25$

S.A. = 25.0

% Coarse = 54.7 ——— Correction = 0.09

$K_m = 1.05 + 0.09$

$= 1.14$

From Fig. V-5 (34)

S.A. = 25.0 S.G. = 2.65 K_m = 1.14

——— Oil Ratio = 3.85%

From Fig. V-6 (34)

Asphalt Pen. 70 at 77°F

Surface Area 25.0

Oil ratio 3.9 ——— Bit. Ratio = 5.4

5.4 lbs. of asphalt are required for 100 lbs. of aggregate

$$\text{Percent of total mix} = \frac{5.4}{(100 + 5.4)} \times 100$$

$$= 5.12$$

Estimated optimum asphalt content = 5.12

Compact Hveem specimens at 5.0, 5.5 and 6.0%

Table 8

Asphaltic Concrete Testing--Hveem Method

(a) Loose Mixture Specific Gravity

Identification	5.0A	5.0B	5.0C	5.5A	5.5B	5.5C	6.0A	6.0B	6.0C
Sample in air, gm	578.0	577.2	569.3	582.2	608.0	558.0	582.0	580.0	581.1
Pyc. + Water, gm	2704.5	2704.5	2704.5	2716.2	2716.2	2716.2	2716.2	2716.2	2716.2
Sum, gm	3282.5	3281.7	3273.8	3298.4	3324.2	3274.2	3298.2	3296.2	3297.3
Pyc. + Sample + Water	3048.2	3041.2	3036.7	3054.6	3070.1	3039.0	3052.3	3051.1	3050.0
Vol. of Sample, cc	243.3	240.5	237.1	243.8	254.1	235.2	245.9	245.1	247.3
Loose Sp. Gr. gm/cc	2.470	2.400	2.403	2.390	2.393	2.375	2.369	2.367	2.350
Theoretical Sp. Gr.	2.445	2.445	2.445	2.435	2.435	2.435	2.414	2.414	2.414

Table 8 (continued)

(b) Specimen Density and Voids

Identification	5.0A	5.0B	5.0C	5.5A	5.5B	5.5C	6.0A	6.0B	6.0C
Wt. in air, gm	1195.5	1198.2	1200.2	1193.2	1191.6	1182.1	1186.1	1185.8	1187.7
Wt. in water	681.2	682.3	679.2	691.2	688.8	680.7	687.8	686.6	685.7
Vol. cc	514.3	515.9	521.0	502.0	502.8	501.4	498.3	499.2	502.0
Density, gm/cc	2.321	2.321	2.305	2.380	2.373	2.360	2.379	2.373	2.365
Density, pcf.	145.0	145.0	144.0	148.5	148.1	147.3	148.5	148.2	147.7
Void Content, %	5.0	5.0	5.7	2.2	2.7	3.1	1.4	1.6	2.0

Table 8 (continued)

(c) Hveem Stabilities

Identification	5.0A	5.0B	5.0C	5.5A	5.5B	5.5C	6.0A	6.0B	6.0C
Spec. Height, in.	2.52	2.53	2.57	2.45	2.47	2.49	2.43	2.44	2.46
Initial Displ. 0.001 in.	202	202	202	202	202	202	200	200	200
P_L at P_v of 3000 lb.	14	13	12	15	14	38	19	25	26
P_L at P_v of 4000 lb.	16	16	14	19	16	52	24	33	33
P_L at P_v of 5000 lb.	19.5	18	16	23	19	69	31	41	43
P_L at P_v of 6000 lb.	23.5	21	18	29	24	86	38	52	55
Final Displ., .001 in.	350	355	390	308	367	338	305	274	290
Specimen Stability, %	55	57	57	54	55	24*	46	41	39
Corrected Stability, %	55	57	57	55	55	24	46	41	39
Cohesiometer Load, gm	664	858	611	1400	1401	1118	1425	1143	1213
Cohesiometer Value	212	273	190	464	460	362	478	371	400

*This value was omitted.

APPENDIX B

CHART SHOWING THE TEST VARIABLES REPRESENTED IN THIS STUDY

Constants: Thickness, Gradation, Compactive Effort

SUPPORT	0.0 psi			1.0 psi									2.0 psi		
TEMPERATURE	59	75	90	59			75			90			59	75	90
ASPHALT	5.5			5.0	5.5	6.0	5.0	5.5	6.0	5.0	5.5	6.0	5.5		

Replicate specimens were made for each test variable.

APPENDIX C

TYPICAL DATA RECORDED FROM THE DEFLECTOMETER TESTS

DEFLECTOMETER DATA SHEET

Mix. I-19 A.C. 6.0% Thick. 1.95 in. D.L. 175# L.L. 150# Mass 4 disc Date: Mold 1/25/72
Speed 740 rpm Press. 1.0 psi Cont. Area 5.0 in^2 Temp. 75°F Spec. 6.0-1 Test 2/4/72

Density	Counter Reps.		4" L D d	2 1/4C D d	Load Disc	Press Gage	Time and Remarks
Before Test	0	0	300	700	0.000	1.00	10:27 am
Wt. (air) 17,770 gm	50	1,000	315.5	662 -663	84.5 -92.0	1.68	
Wt. (H_2O) 10,070 gm	200	4,000	19.0	51.5 -52.0	106.5 -113.0	1.85	
Vol. 7,700 cc	500	10,000	20.5	46.0 -46.5	119.5 -126.0	2.00	
Density 2.308 gm/cc	1,000	20,000	21.0	42.5 -43.5	128.5 -134.5	2.07	
Max. S.G. 2.414 gm/cc	2,500	50,000	21.5	41.5 -42.0	139.5 -146.0	2.17	
Void 4.39 %	5,000	100,000	21.0	35.5 -36.5	149.0 -155.5	2.20	
	11,500	230,000	21.0	30.0 -30.5	167.0 -173.5	2.29	3.44 pm
	16,000	320,000	21.5	26.5 -27.0	180.5 -187.0	2.29	5.47 pm
	20,000	400,000	21.5	623 -624	194.5 -201.0	2.31	7.38 pm

APPENDIX D

REDUCED DATA FROM DEFLECTOMETER TESTS

Table 9

Mixture Characteristics

Specimen No*	Density, gm/cc	Void Content, %	Height, in.
5.0 -1	2.323	4.99	1.92
5.0 -2	2.316	5.28	1.93
5.5 -1	2.231	8.38	2.03
5.5 -2	2.243	7.89	2.06
5.5 -3	2.318	4.80	1.94
5.5 -4	2.315	4.93	1.94
5.5 -5	2.369	2.71	1.88
5.5 -6	2.347	3.61	1.91
5.5 -7	2.290	5.95	1.98
5.5 -8	2.312	5.05	1.93
6.0 -1	2.308	4.39	1.95
6.0 -2	2.300	4.72	1.96
5.5 -9	2.286	6.12	1.97
5.5 -10	2.285	6.16	1.96
5.5 -11	2.320	4.72	1.93
5.5 -12	2.255	7.39	2.00
5.5 -13	2.323	4.60	1.91
5.5 -14	2.310	5.13	1.95
5.0 -3	2.296	6.09	1.94

*Specimens are numbered to indicate asphalt content and series number for that asphalt content.

Table 9 (continued)

Mixture Characteristics

Specimen No	Density, gm/cc	Void Content, %	Height, in.
5.0 -4	2.281	6.71	1.96
6.0 -3	2.296	4.89	1.96
6.0 -4	2.365	1.89	1.87
5.0 -5	2.277	6.87	1.99
5.0 -6	2.247	8.10	2.01
5.5 -15	2.358	3.16	1.88
5.5 -16	2.319	4.74	1.93
5.5 -17	2.314	4.97	1.95
5.5 -18	2.286	6.12	1.98
5.5 -19	2.298	5.63	1.95
5.5 -20	2.331	4.27	1.92
6.0 -5	2.329	3.52	1.93
6.0 -6	2.280	5.55	2.00

Table 10

Deflectometer Test Data

Temperature, °F	Load Disc. in^2	Specimen No	Support Pressure, psi		
			Initial	Final, P_f	Mean, P_m
41	5.00	5.5-9	1.00	--	1.50
59	3.142	5.5-10	1.00	1.90	1.50
		5.5-11	0.10	1.88	1.00
		5.5-12	0.10	1.70	1.00
		5.5-13	2.00	2.60	2.50
		5.5-14	2.00	2.60	2.50
		5.0-3	1.00	2.39	1.50
		5.0-4	1.00	2.05	1.50
		6.0-3	1.00	2.18	1.50
		6.0-4	1.00	1.88	1.50
75	5.00	5.0-1	1.00	2.65	1.50
		5.0-2	1.00	2.28	1.50
		5.5-1	1.00	2.70	1.50
		5.5-2	1.00	2.57	1.50
		5.5-3	1.00	2.00	1.50
		5.5-4	1.00	2.00	1.50
		5.5-5	0.00	1.78	1.00
		5.5-6	0.00	1.64	1.00
		5.5-7	2.00	2.68	2.50
		5.5-8	2.00	3.00	2.50

Table 10 (continued)

Deflectometer Test Data

Temperature, °F	Load Disc. in^2	Specimen No	Support Initial	Pressure, psi Final, P_f	Mean, P_m
		6.0-1	1.00	2.20	1.50
		6.0-2	1.00	2.40	1.50
90	5.00	5.0-5	1.00	2.60	1.50
		5.0-6	1.00	2.70	1.50
		5.5-15	1.00	2.55	1.50
		5.5-16	1.00	2.58	1.50
		5.5-17	0.10	2.25	1.00
		5.5-18	0.30	1.94	1.00
		5.5-19	2.00	3.00	2.50
		5.5-20	2.00	3.20	2.50
		6.0-5	1.00	2.72	1.50
		6.0-6	1.00	2.75	1.50

Table 11

Deflectometer Test Data Computation

Specimen No	Repeated central deflection d (x10^{-3} in.)		Specimen thickness, t, in.	t^2	δt^3
	At P_m	At P_f			
5.5-9	5.5	--	1.97	3.881	42.051
5.5-10	8.0	6.5	1.96	3.842	48.947
5.5-11	8.5	7.0	1.93	3.725	50.325
5.5-12	8.0	7.5	2.00	4.000	60.000
5.5-13	9.0	8.0	1.91	3.648	55.741
5.5-14	9.5	7.5	1.95	3.803	55.619
5.0-3	7.0	6.5	1.94	3.764	47.464
5.0-4	7.5	7.5	1.96	3.842	56.477
6.0-3	6.0	4.5	1.96	3.842	33.886
6.0-4	5.0	4.5	1.87	3.497	29.427
5.0-1	6.0	5.0	1.92	3.686	35.386
5.0-2	6.0	6.0	1.93	3.725	43.136

Table 11 (continued)

Deflectometer Test Data Computation

Specimen No	Repeated central deflection d ($\times 10^{-3}$ in.)		Specimen thickness, t, in.	t^2	St^3
	At P_m	At P_f			
5.5-1	2.5	5.0	2.03	4.121	41.828
5.5-2	6.0	7.5	2.06	4.244	65.570
5.5-3	6.0	6.0	1.94	3.764	43.813
5.5-4	7.0	5.0	1.94	3.764	36.511
5.5-5	5.0	5.0	1.88	3.534	33.220
5.5-6	6.0	6.5	1.91	3.648	45.290
5.5-7	6.0	5.5	1.98	3.920	42.689
5.5-8	11.0	12.0	1.93	3.725	86.271
6.0-1	7.5	6.5	1.95	3.803	48.203
6.02	5.5	5.0	1.96	3.842	37.652
5.0-5	11.0	11.5	1.99	3.960	90.625
5.0-6	10.0	12.5	2.01	4.040	101.505

Table 11 (continued)

Deflectometer Test Data Computation

Specimen No	Repeated central deflection d ($\times 10^{-3}$ in.)		Specimen thickness, t, in.	t^2	$\mathcal{S}t^3$
	At P_m	At P_f			
5.5-15	7.0	8.0	1.88	3.534	53.151
5.5-16	10.0	11.0	1.93	3.725	79.082
5.5-17	7.0	7.0	1.95	3.803	51.911
5.5-18	5.0	5.5	1.98	3.920	42.689
5.5-19	6.5	7.0	1.95	3.803	51.911
5.5-20	7.0	6.5	1.92	3.686	46.001
6.0-5	7.0	6.5	1.93	3.725	46.730
6.0-6	7.0	7.5	2.00	4.000	60.000

Table 12

Dynamic Stiffness, Central Deflections, and Strains

Specimen No	Dynamic Stiffness, psi		Radial Strain (10^{-3})		Central deflection at failure, in.
	At P_m	At P_f	At P_m	At P_f	
5.5-9	68,726	--	1.100	--	(0.153)*
5.5-10	60,596	56,124	1.465	1.531	0.180
5.5-11	63,527	55,062	1.491	1.615	0.220
5.5-12	53,033	47,733	1.659	1.760	0.195
5.5-13	45,998	44,678	1.911	1.943	0.112
5.5-14	46,369	44,503	1.822	1.867	0.152
5.0-3	61,836	53,417	1.459	1.588	0.155
5.0-4	52,234	48,034	1.696	1.778	0.190
6.0-3	85,729	75,701	1.026	1.103	0.151
6.0-4	97,699	91,576	0.985	1.022	0.170
5.0-1	82,095	67,188	0.971	1.079	0.139
5.0-2	67,345	59,526	1.172	1.248	0.152

*The specimen did not fail.

Table 12 (continued)

Dynamic Stiffness, Central Deflections, and Strains

Specimen No	Dynamic Stiffness, psi		Radial Strain (10-3)		Central deflection at failure, in.
	At P_m	At P_f	At P_m	At P_f	
5.5-1	66,893	56,324	1.049	1.147	0.201
5.5-2	44,304	37,947	1.564	1.692	0.100
5.5-3	66,305	61,358	1.178	1.225	0.136
5.5-4	80,387	72,808	0.975	1.027	0.138
5.5-5	93,016	82,881	0.923	0.977	0.187
5.5-6	68,911	63,135	1.213	1.266	0.173
5.5-7	57,931	55,745	1.198	1.223	0.100
5.5-8	30,427	28,276	2.458	2.553	0.185
6.0-1	61,200	54,302	1.271	1.352	0.159
6.0-2	76,756	66,012	0.994	1.076	0.162
5.0-5	33,732	28,657	2.250	2.443	0.174
5.0-6	29,821	25,459	2.485	2.683	0.210
5.5-15	55,220	47,252	1.512	1.638	0.180

Table 12 (continued)

Dynamic Stiffness, Central Deflections, and Strains

Specimen No	Dynamic Stiffness, psi		Radial Strain (10^{-3})		Central deflection at failure, in.
	At P_m	At P_f	At P_m	At P_f	
5.5-16	38,277	32,756	2.100	2.241	0.230
5.5-17	60,700	50,301	1.326	1.457	0.273
5.5-18	72,384	63,234	1.071	1.143	0.196
5.5-19	47,928	44,060	1.496	1.564	0.115
5.5-20	54,412	47,510	1.362	1.469	0.176
6.0-5	62,808	51,207	1.263	1.404	0.247
6.0-6	48,917	40,173	1.509	1.670	0.250

Table 13

Radial Stresses and Load Repetitions to Failure

Specimen No	Load Repetitions N_f ($\times 10^4$)	Radial Stress, psi At P_m	Radial Stress, psi At P_f	Stress equivalent for 5 in^2 disc At P_m	Stress equivalent for 5 in^2 disc At P_f
5.5-9	800	75.6	--		
5.5-10	230	88.8	85.9	77.2	74.4
5.5-11	120	94.7	88.9	82.7	77.0
5.5-12	140	88.0	84.0	76.9	72.9
5.5-13	110	87.9	86.8	75.6	74.7
5.5-14	150	84.5	83.1	72.6	71.4
5.0-3	90	90.2	84.8	78.4	73.1
5.0-4	190	88.6	85.4	77.0	73.9
6.0-3	110	88.0	83.5	76.5	72.1
6.0-4	28	96.2	93.6	83.7	81.1
5.0-1	60	79.7	72.5		
5.0-2	7	78.9	74.3		
5.5-1	10	70.2	64.6		
5.5-2	19	69.3	64.2		
5.5-3	9.5	78.1	75.2		
5.5-4	15	78.4	74.8		

Table 13 (continued)

Radial Stresses and Load Repetitions to Failure

Specimen No	Load Repetitions N_f ($\times 10^4$)	Radial Stress, psi At P_m	At P_f
5.5-5	30	85.9	81.0
5.5-6	17	83.6	79.9
5.5-7	13	69.4	68.2
5.5-8	30	74.8	72.2
6.0-1	12	77.8	73.4
6.0-2	14	76.3	71.0
5.0-5	1.0	75.9	70.0
5.0-6	1.1	74.1	68.3
5.5-15	1.4	83.5	77.4
5.5-16	1.9	80.4	73.4
5.5-17	0.9	80.5	73.3
5.5-18	1.0	77.5	72.3
5.5-19	0.44	71.7	68.9
5.5-20	1.4	74.1	69.8
6.0-5	1.9	79.3	71.9
6.0-6	0.83	73.8	67.1

Table 14

Load Repetitions Reduced to a Stress-Level of 80 psi

Specimen No.	Equivalent N_f ($\times 10^4$)	
	5 in^2 disc.	80 psi.
5.5-9	--*	--*
5.5-10	400	350
5.5-11	200	230
5.5-12	240	205
5.5-13	185	150
5.5-14	260	180
5.0-3	160	150
5.0-4	330	290
6.0-3	190	165
6.0-4	47	55
5.0-1	60	59
5.0-2	7	6.7
5.5-1	10	6.1
5.5-2	19	10.7
5.5-3	9.5	8.7
5.5-4	15	14

*Specimen did not fail.

Table 14 (continued)

Load Repetitions Reduced to a Stress-Level of 80 psi

Specimen No.	Equivalent N_f ($\times 10^4$)	
	5 in^2 disc.	80 psi.
5.5-5	30	39.5
5.5-6	17	20.5
5.5-7	13	7.4
5.5-8	30	23
6.0-1	12	10.7
6.0-2	14	11
5.0-5	1.0	0.8
5.0-6	1.1	0.8
5.5-15	1.4	1.6
5.5-16	1.9	1.9
5.5-17	0.9	0.91
5.5-18	1.0	0.90
5.5-19	0.44	0.28
5.5-20	1.4	1.0
6.0-5	1.9	1.8
6.0-6	0.83	0.61

REFERENCES

1. Monismith, C. L., "Asphalt Paving Mixtures, Properties, Design, and Performance," Prepared for the Short Course in Asphalt Paving, 1961-62, Berkeley, California.

2. Finn, F. N., "Factors Involved in the Design of Asphaltic Pavement Surfaces," National Cooperative Highway Research Program Report No. 39, 1967.

3. Hveem, F. N., and Carmany, R. M., "The Factors Underlying the Rational Design of Pavements," Proceedings, Highway Research Board, Vol. 28, 1948; pp. 101-136.

4. Hveem, F. N., "Pavement Deflections and Fatigue Failures," Bulletin 114, Highway Research Board, 1955; pp. 43-87.

5. Pell, P. S., "Fatigue of Asphalt Pavement Mixes," Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, 1967; pp. 577-593.

6. Monismith, C. L., "Flexibility Characteristics of Asphalt Paving Mixtures," Proceedings, Association of Asphalt Paving Technologists, Vol. 27, 1958.

7. Papazian, H. S., and Baker, R. F., "Analysis of Fatigue Type Properties of Bituminous Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 28, 1959.

8. Monismith, C. L., Secor, K. E., and Blackmer, E. W., "Asphalt Mixture Behavior in Repeated Flexure," Proceedings, Association of Asphalt Paving Technologists, Vol. 30, 1961.

9. Jimenez, R. A., "An Apparatus for Laboratory Investigations of Asphaltic Concrete Under Repeated Flexural Deformations," Report submitted to the Texas Highway Department, 1962.

10. Vallerga, B. A., Finn, F. N., and Hicks, R. G., "Effects of Asphalt Aging on the Fatigue Properties of Asphalt Concrete," Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, 1967; pp. 595-617.

11. Jimenez, R. A., "Aspects of the Bending Strength of Asphaltic Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 36, 1967.

12. Layman, A. H., and Phillipi, S. W., "A Study of the Flexural Properties of Certain Black Base Mixtures," Proceedings, Association of Asphalt Paving Technologists, Vol. 38, 1969; pp. 684-705.

13. Rader, L. F., "Correlation of Low Temperature Tests with Resistance to Cracking of Sheet Asphalt Pavements," Proceedings, Association of Asphalt Paving Technologists, January, 1936.

14. Rader, L. F., "Report on Further Research Work on Correlation of Low Temperature Tests with Resistance to Cracking of Sheet Asphalt Pavements," Proceedings, Association of Asphalt Paving Technologists, January, 1937.

15. Thomas, T. W., "The Testing of Asphalt Paving Specimens Upon a Flexible Spring Base," Proceedings, Association of Asphalt Paving Technologists, Vol. 17, 1948.

16. Hughes, E. C., and Faris, R. B., Jr., "Low Temperature Maximum Deformability of Asphalts," Proceedings, Association of Asphalt Paving Technologists, Vol. 19, 1950; pp. 329-347.

17. Wood, L. E., and Goetz, W. H., "The Strength of Bituminous Mixtures and their Behavior Under Repeated Loads," Proceedings, Highway Research Board, Vol. 35, 1956; pp. 405-417; Vol. 36, 1957; pp. 318-333.

18. Goetz, W. H., McLaughlin, J. F., and Wood, L. E., "Loading Deformation Characteristics of Bituminous Mixtures Under Various Conditions of Loading," Proceedings, Association of Asphalt Paving Technologists, Vol. 26, 1957.

19. Monismith, C. L., "Effect of Temperature on the Flexibility Characteristics of Asphaltic Paving Mixtures," Proceedings, Third Pacific Area Meeting, ASTM, October 1959.

20. Deacon, J. A. and Monismith, C. L., "Laboratory Flexural-Fatigue Testing of Asphalt-Concrete with Emphasis on Compound-Loading Tests," Record No. 158, Highway Research Board, 1967; pp. 1-31.

21. Seed, Mitry, Monismith, and Chan, "Prediction of Flexible Pavement Deflections from Laboratory Repeated-Load Tests," National Cooperative Highway Research Program Report No. 35, 1967.

22. Heukelom, W. and Klomp, A. J. G., "Road Design and Dynamic Loading," Proceedings, Association of Asphalt Paving Technologists, Vol. 33, 1964; pp. 92-125.

23. Epps, J. A. and Monismith, C. L., "Influence of Mixture Variables on the Flexural Fatigue Properties of Asphalt Concrete," Proceedings, Association of Asphalt Paving Technologists, Vol. 38, 1969; pp. 423-464.

24. Baker, R. F. and Papazian, H. S., "The Effect of Stiffness Ratio on Pavement Stress Analysis," Proceedings, Highway Research Board, Vol. 39, 1960; pp. 61-85.

25. Dehlen, G. L., "Flexure of a Road Surfacing, Its Relation to Fatigue Cracking, and Factors Determining its Severity," Bulletin 321, Highway Research Board, 1962; pp. 26-39.

26. Jimenez, R. A., "Fatigue Testing of Asphaltic Concrete-Slabs," A paper for Presentation at the ASTM Symposium on Fatigue of Compacted Bituminous Aggregate Mixtures, July, 1971.

27. Kondner, R. L. and Krizek, R. J., "Factors Influencing Flexible Pavement Performance," National Cooperative Highway Research Program Report No. 22, 1966.

28. White, O. A., "Fatigue and Deflection of Asphaltic Concrete," Record No. 131, Highway Research Board, 1966; pp. 154-159.

29. Metcalf, C. T., "Flexural Tests of Paving Materials for Orthotropic Steel Plate Bridges," Record No. 155, Highway Research Board, 1967; pp. 61-81.

30. Bazin, P. and Saunier, J., "Deformability, Fatigue, and Healing Properties of Asphalt Mixes," Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, 1967; pp. 553-570.

31. Kirk, J. M., "Results of Fatigue Tests on Different Types of Bituminous Mixtures," Proceedings, Second International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, 1967; pp. 571-575.

32. Pell, P. S. and Taylor, I. F., "Asphaltic Road Materials in Fatigue," Proceedings, Association of Asphalt Paving Technologists, Vol. 38, 1969; pp. 371-422.

33. Schmidt, R. J., and Santucci, L. E., "The Effect of Asphalt Properties on the Fatigue Cracking of Asphalt Concrete on the Zaca-Wigmore Test Project," Proceedings, Association of Asphalt Paving Technologists, Vol. 38, 1969.

34. The Asphalt Institute: Mix Design Methods For Asphalt Concrete and Other Hot-Mix Types, 1963.