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FINAL REPORT

DEVELOPMENT OF EQUIPMENT AND TECHNIQUES FOR EVALUATING FATIGUE AND RUTTING CHARACTERISTICS OF ASPHALT CONCRETE MIXES

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DEVELOPMENT OF EQUIPMENT AND TECHNIQUES FOR EVALUATING FATIGUE AND RUTTING CHARACTERISTICS OF ASPHALT CONCRETE MIXES

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ABSTRACT

Fatigue and rutting characteristics are evaluated for a range of asphalt concrete base mixes. A rectangular beam on a rubber subgrade is used in the fatigue test. Tensile strain in the beam is measured by means of a strain gage bonded to the beam. Rutting characteristics are determined using both the Shell creep test and also a repeated load triaxial test. A temperature of 95°F was used which is the mean pavement temperature for rutting in Georgia.

The fatigue life of a mix was found to be primarily dependent upon the void and asphalt content of the mix. Fatigue life is inversely proportional to the air voids on a log-log plot with a small change in air voids having a large influence on fatigue life. For a long fatigue life the air void content should be between 2 and 4 or 5%. Going from an asphalt content of 4.5 to 4.75% approximately doubled the fatigue life. Other variables studied included mineral filler, asphalt cement viscosity, aggregate type and gradation, and 50 and 75 blow Marshall mixes.

Rutting tests indicated that moderate changes in mix variables do not greatly affect rutting.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

LIST OF FIGURES (Continued)

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CHAPTER 1

INTRODUCTION

An asphalt concrete pavement should be designed to provide a durable, skid resistant surface which is both stable and resistant to a fatigue type failure under in-service conditions. When subjected to large numbers of heavy wheel loadings, a stable surfacing does not undergo either an objectionable amount of shoving or an objectionable accumulation of permanent deformation under traffic. Fatigue resistance is the ability to withstand cracking of stabilized layers due to repeated flexing of the surfacing that occurs with the passage of a large number of heavily loaded vehicles over the pavement. The structural design of a pavement consists of selecting compatible combinations of materials and layer thicknesses which minimize the occurrence of both fatigue and rutting type failures in the pavement.

A fatigue failure of a stabilized surface or base course results in cracking which in turn allows water to enter the pavement structure. As a result the load spreading capability of the pavement is reduced which together with the detrimental effects of the water can eventually lead to serious rutting in the base, subbase, and subgrade. A previous study by Barksdale [1] has clearly indicated that fatigue distress is particularly likely to occur in pavement systems constructed over the highly micaceous, silty sand subgrade typically found in the Piedmont Province of Georgia. Fatigue can also be an important distress mode in the other geologic areas of Georgia for all types of flexible pavement sections.

The asphalt concrete surface and base course are critical components of a flexible pavement structure, and it is essential to minimize cracking and rutting in these layers. To meet varying needs, the Georgia Department of Transportation uses several different asphalt concrete surface and base course mixes. Laboratory investigations [2-5] have indicated that the fatigue and durability performance of an asphalt concrete mix is significantly influenced by the asphalt content, percent voids, mineral filler, the characteristics of bitumen binder, and to a lesser extent by several other variables. The effects of these variables on the fatigue and rutting performance of the asphalt concrete mixes used in Georgia have not been previously determined through distinct laboratory studies. Furthermore, at the present time asphalt concrete mixes are usually designed by the Georgia Department of Transportation using the Marshall mix design method more for stability than for fatigue resistance.

Project Objectives

The purpose of this project is to develop practical laboratory test methods that can be used to maximize fatigue resistance of asphalt concrete base course mixes currently used while at the same time provide a sufficient degree of stability of the mix to avoid excessive rutting. The influence of pertinent variables were also investigated on the fatigue and rutting performance of selected base course mixes.

The specific objectives of this investigation can be summarized as follows:

1. To develop a practical fatigue test and testing procedure which can be used in research and/or for standard asphalt concrete base course mix designs.

2. To develop and recommend mix design criteria to optimize the fatigue life of the mix, while at the same time obtaining a mix which will remain stable and not undergo excessive amounts of rutting.

3. To evaluate the relative influence of selected mix variables on the fatigue and rutting characteristics of the black base and modified B base course mixes presently used by the Georgia Department of Transportation.

The above objectives are accomplished by developing a fatigue test using a rectangular shaped beam supported on a rubber (elastic) subgrade. A repeated load is applied at the center of the beam until fatigue failure occurs. The initial tensile strain in the beam is measured by a strain gauge bonded to the beam. The fatigue tests were performed at $80^{\circ}F$ ($27^{\circ}C$) in an environmental chamber. The tests developed for evaluating rutting consist of both a simple creep test proposed by Shell Amsterdam which is performed on an unconfined specimen, and also a repeated load triaxial test performed at a single confining stress. Both tests are conducted at a temperature of $90^{\circ}F$ ($35^{\circ}C$) which is approximately the theoretical mean temperature for rutting of asphalt concrete pavements in Georgia consisting of approximately 10.5 in. (267 mm) of asphalt concrete [47].

A comprehensive study is made of the rutting and fatigue characteristics of both black base and modified B base course mixes. Primary mix variables investigated include asphalt content, voids content and 50 and 75 blow Marshall Mix Designs using a granitic gneiss aggregate. The effects are also studied of material gradation, aggregate type, mineral filler and asphalt viscosity of AC-20 and AC-40 on fatigue and rutting performance. Preliminary mix design criteria are presented for maximizing fatigue life, while limiting rut depth to an acceptable level. The results of the laboratory tests are used to predict pavement performance in terms of rut depth and fatigue life of a typical pavement section.

CHAPTER 2

LITERATURE REVIEW

Introduction

Predicting the fatigue life and rut depth in asphalt concrete pavements is a complicated problem which is still in the developmental stage. A nationwide survey [6] of pavement performance indicates that fatigue is a much more common type of distress mechanism than rutting. As a result of this fact, fatigue of asphalt concrete mixes has been investigated quite extensively over the past fifteen years. Much of this work has been summarized in the publication STRUCTURAL DESIGN OF ASPHALT CONCRETE PAVEMENTS TO PREVENT FATIGUE CRACKING(1). In contrast, prediction of rut depth in asphalt concrete mixes has only been given extensive consideration in approximately the last five years due at least partly to a trend toward higher wheel loads.

Numerous comprehensive investigations have been conducted to study the characteristics of flexible pavement materials and the performance of pavement structures [c.f. 1, 2,3,5]. These studies have clearly shown that pavement performance under service conditions is affected by both the characteristics of the materials in the individual layers and also by complicated interaction between each layer in the pavement structure. The fatigue performance of asphalt concrete pavements can be approximately predicted using experimentally determined fatigue curves and layered system theory [1,2,5].

Fatigue of Asphalt Concrete

Failure Mechanisms

The fatigue failure of asphalt concrete is caused primarily by the repeated bending due to the passage of large numbers of heavily loaded vehicles over the pavement. Tensile strain caused by moisture and temperature gradients, weathering and aggregate stripping also contribute to fatigue of inservice pavements. The classical type of fatigue failure has often been described by some researchers as a "chickenwire" or "alligator" pattern of cracks which appear on the surface. In at least Georgia, however, longitudinal cracking usually develops first along the edge of the depressed area parallel to the direction of vehicle movement. This cracking may continue to develop with increasing numbers of wheel repetitions so that eventually an alligator pattern may become evident.

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1. Special Report 140, Transportation Research Board, 1973, 201 p.

As a wheel load moves over a pavement, the bottom of the asphalt concrete layer is subjected to a compression-tension-compression load cycle while the top of the layer is subjected to the reverse cycle. Direct tension tests conducted by Raithby and Sterling [8] indicate a significant difference in fatigue life may exist depending upon the type of stress reversal which occurs before a rest period. The tests conducted by Raithby and Sterling indicate that going from tension to compression (similar to the condition occurring in the top of the pavement) resulted in approximately twice the fatigue life which occurred when going from compression to tension. These results indicate that the fatigue resistance due to wheel loadings of the top and bottom of the asphalt concrete layer may be different even if all other factors were the same.

Initial fracture of the asphalt concrete has been often considered by researchers to initiate in the bottom of the asphalt concrete layer. However, a comprehensive investigation of pavement distress conducted by the Georgia Department of Transportation [7] showed that on I-285 in one instance the cracks in the wheel path initiated in the asphalt concrete surfacing. At two other locations on I-285, at the time of the investigation cracks had extended through all the asphalt concrete courses so that the location of crack initiation could not be established. Further, Baker and Quinn [9] found in New Jersey that crack initiation in one project also began in the surface course.

Although theory as presently applied appears to indicate that failure should occur in the bottom of the layer, many complicating factors such as construction variables, braking forces, temperature and moisture gradients and stress reversals have not yet been taken into consideration. Also, rutting causes tensile strains in the top of the asphalt concrete layer immediately adjacent to the wheel path. These tensile strains would add to those caused by a wheel load, and are probably another contributing factor to the initiation of fracture in the top of the asphalt concrete layer. The conclusion can thus be reached that crack initiation can apparently begin in either the top or bottom of the asphalt concrete layer.

Controlled Stress and Controlled Strain Testing

Laboratory fatigue tests have been performed using both controlled stress and controlled strain type of loading. Laboratory studies by Monismith and Deacon [3] have indicated that in a thin asphalt concrete surfacing less than approximately 2 in. (51 mm) in thickness, the controlled strain test is probably more representative of field conditions. On the other hand, when the total thickness of the asphalt concrete layers is greater than approximately 6 in. (152 mm), the controlled stress testing condition is more appropriate. The *total thickness of all asphalt concrete layers contributing structural strength should be considered* in selecting whether controlled stress or controlled strain tests are appropriate. For intermediate conditions Monismith and Deacon have proposed using a mode factor in characterizing the fatigue life of asphalt concrete layers to permit a gradual transition from the controlled strain to the controlled stress type of loading with increase in thickness. The controlled strain mode of testing has been found to indicate a greater fatigue life than controlled stress. From a study of the AASHO Road Test results, Barksdale [1] has concluded that the fatigue resistance of a structural layer decreases significantly with increase in thickness which is in agreement with the mode factor concept.

In the controlled stress test the formation of the initial crack is quickly followed by rapid crack propagation and complete specimen failure. On the other hand in the controlled strain test, when the crack initiates, to maintain the same strain level a reduction in stress occurs around the crack. Because of the reduction in stress which occurs as the crack length increases, crack propagation is relatively slow compared with the controlled stress mode of failure. The difference in fatigue life between the two tests is mostly accounted for by the different rate of crack propagation, with the fatigue life in the controlled strain test being greater than for the controlled stress test.

From the above discussion the following three important practical conclusions can be made concerning the controlled stress type of fatigue test:

1. The Georgia DOT at the present time generally uses total asphalt concrete thicknesses greater than 6 in. (152 mm) on primary and interstate highways. Therefore, the controlled stress mode of fatigue testing is appropriate for investigating the fatigue characteristics of mixes to be used in at least primary and interstate highway construction.
2. For the controlled stress test, fatigue failure can be defined for practical purposes as the complete fracture of the specimen since crack propagation is rapid in this type test.
3. The controlled stress test gives conservative fatigue test results for layer thicknesses less than that required for controlled stress conditions.

Test Methods

Important considerations in selecting a suitable laboratory fatigue test method for routine use in a mix design method are as follows: (1) the test should indicate a fatigue life that is comparable to that which should be developed in the field under actual dynamic stress conditions, (2) specimen preparation should not be difficult, (3) the test apparatus should be relatively simple and (4) the test should give reproducible results so as to minimize the required amount of testing. In general, to give a fatigue life that is comparable to that developed in the field, the test method should as closely as practical duplicate the conditions of loading, support and stress state to which the material is subjected in the pavement.

Bonnot [29] has found that a beam specimen subjected to bending has a fatigue life at least 50 percent greater than obtained in uniaxial tension. Further, their studies indicate that loaded slabs have fatigue lives that are even higher than for beam bending tests. These results indicate the importance of duplicating at this time the actual stress state that will be developed in the pavement.

Fatigue characteristics of asphalt concrete mixes have been evaluated in the laboratory using beam, circular slab, trapezoidal or hyperbolic shape specimens [5]. These specimens have been loaded using several different loading arrangements, and have been tested in both the unsupported [3-5,8,10-19] and supported conditions [1,19-21]. Several investigators have used rectangular, trapezoidal, or hyperbolic shaped specimens tested as unsupported cantilever beams [5,11,12,15-17] subjected to sinusoidal loadings. Direct [8,18] and indirect (diametral) [22] tension tests have also been used to characterize the fatigue behavior of asphalt concrete specimens. Simply supported rectangular beam specimens subjected to bending [3,4,10,13,14] have also been frequently used.

Bazin and Saunier [23] have suggested that thick pavements tend to induce small internal stresses that promote healing of fatigue cracks, thereby prolonging fatigue life under conditions of rest periods. That healing can actually occur in the field has been demonstrated on a test pavement in New Jersey [9]. Several investigators [4,8,23,24] have found in laboratory tests that the presence of a rest period after each load pulse can significantly increase fatigue life. Specimens subjected to a continuous sinusoidal loading were found by Raithby and Sterling [8] to have fatigue lives approximately 25 times smaller than similar specimens subjected to individual load pulses having a 1 sec. rest period between each pulse (Fig. 1). The effect of rest period was found to become less at temperatures above 25^{o}C (77^{o}F) and to be dependent upon the stress level. Raithby and Sterling concluded that for the mixes investigated and load pulses of 0.4 sec. duration, a rest period greater than 1 sec. was sufficient to develop the maximum fatigue

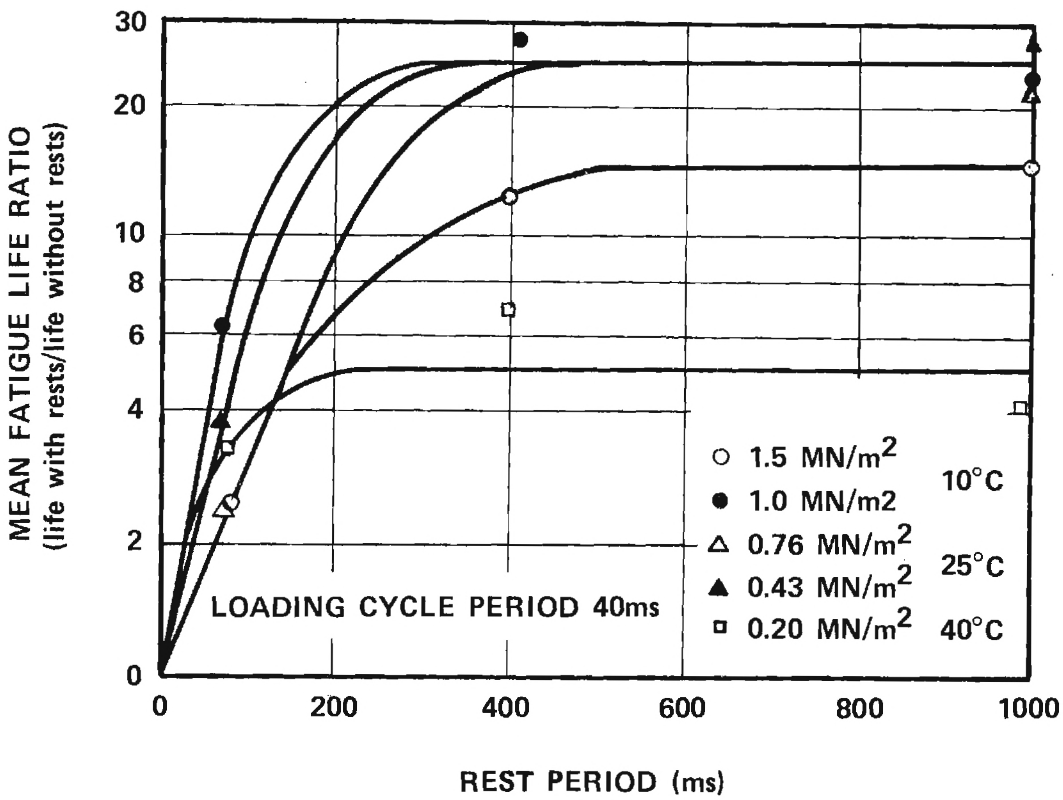


FIGURE 1. INFLUENCE OF REST DURATION ON FATIGUE LIFE RATIO (AFTER RAITHBY AND STERLING, REF. 8)

life. These findings are in agreement with those of Van Dijk and Visser [25].

In a pavement a rest period occurs after the application of each load pulse sequence. Therefore, to obtain fatigue lives that are comparable with those that would be expected to occur in the field, the test method used should allow a rest period after each load pulse sequence. Use of a continuous sinusoidal load pulse such as that often used in the past [10-12, 15-17, 18, 22-24] under at least some conditions may greatly underestimate fatigue life although the test can be performed in a shorter period of time. Whether the relative fatigue life of various mixes can be predicted using a sinusoidal loading has not yet been established.

The direct tension fatigue test [8,18] and the diametral fatigue test [22] have both increased in popularity in recent years. The direct tension test consists of subjecting a small rectangular or cylindrical shaped specimen to a uniaxial stress state. This method permits application of stress reversals. The specimens must, however, be glued to end tabs so they can be clamped in a testing machine. The diametral tension test consists of loading a disc shaped specimen in compression at the top and bottom in the direction of its diameter. The applied loading causes a biaxial tension stress state in the specimen. An important practical advantage of the direct tension and diametral test is that small specimens which can be either cut from the pavement or prepared in the laboratory are used in the test. On the other hand, the specimens are not subjected to bending, are tested in an unsupported condition, and are not subjected to the same type stress condition as would exist in the field.

The asphalt concrete in a pavement structure is subjected to a biaxial bending stress state with relatively large vertical stresses occurring perpendicular to the plane of bending. To more nearly duplicate these stress conditions, several researchers have used circular slab specimens supported either on a rubber subgrade [1,20,21] or a cushion of air [19]. A circular shaped load is applied to the center of the slab and cycled until fatigue failure occurs. This type of test results in biaxial tensile stresses in the slab which are very similar to those occurring in the pavement. Usually this test is performed so that stress reversals do not occur. In contrast, the bending stress state developed in a beam fatigue test specimen results in tensile stress in only one direction (uniaxial bending tension). Recent studies [20] have indicated that the fatigue life obtained using the supported circular slab test is greater than that obtained using a rectangular, unsupported beam specimen. Although the circular slab fatigue test is probably the most desirable at this time, the tremendous effort and expense involved in preparing a specimen makes it impractical for use as a routine test.

Most fatigue tests in the United States have been performed on simply supported beams subjected to a two point loading [3,4,10,13,14]. Although this test does induce uniaxial bending in the specimen, the unsupported beam test has the important disadvantage that tensile strains and creep movements can be induced in the specimens due to the weight of the beam. Also, the beam is unsupported over most of its length which is not representative of the actual field condition of support, and the testing apparatus is moderately complex.

The supported beam fatigue test consists of placing a rectangular asphalt concrete beam on an elastic subgrade support usually made of rubber. The advantages of this method include (1) full support of the beam, (2) a stress state similar to that occurring in the field except that uniaxial rather than biaxial bending is developed, (3) a very simple testing apparatus can be used and (4) the rectangular beam specimens can be reasonably easily prepared. Further, support of the specimen would be expected to reduce the effects of minor imperfections in the specimens compared to unsupported beams and hence reduce the scatter of test results.

The major disadvantages of the supported beam test are that a biaxial state of bending stress is not developed in the specimen, and the specimen cannot be subjected to stress reversals and accompanying rotation of principal stresses.

A wheel load passing over a pavement surface actually causes three successively occurring loading pulses rather than the single pulse usually applied in a laboratory fatigue test. In the bottom of the asphalt concrete layer the material first goes into compression as the wheel loading approaches the point. As the wheel moves over the point the material at the bottom goes into tension and then back to compression as the wheel moves away; the pulse sequence in the top of the layer is just the reverse of the bottom. The initial compressive strain pulse in the bottom of the layer has been measured to be approximately 1/7 that of the tensile strain pulse which follows [8]. Raithby and Sterling [8] have found that going from one equal compression-tension pulse loading to just a tension pulse resulted in an increase in fatigue life of approximately 30 percent. For the smaller initial compressive pulses which should occur in a pavement, the difference in fatigue life should be at most on the order of only 10 to 15 percent.

Based on these considerations, the supported beam fatigue test was selected for use in this investigation as a practical method for simulating for routine testing applications reasonably closely the overall stress conditions occurring in an asphalt concrete layer.

Other tests such as the diametral test could also have been used.

Fatigue Test Results

For both constant stress and constant strain tests, numerous studies indicate that the log of the initial tensile strain is approximately proportional to the log of the number of cycles to crack initiation:

$$N_i = C\left(\frac{1}{\varepsilon}\right)^m \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (1)$$

where N_i = number of load applications to crack initiation

ε = amplitude of applied tensile strain

C and m = factors dependent upon the properties of the asphalt concrete mix.

When the results are presented in terms of tensile strain, the effect of stiffness of a given mix is approximately removed as a variable. Since load rate and temperature effect the mix stiffness, the fatigue test can be performed at a single temperature and load rate, and the fatigue curve obtained can be used for other temperatures and load rates. However, as pointed out by Pell [26], some evidence indicates for temperatures above 77^{o}F (25^{o}C) a unique curve may not be obtained. The slope of the fatigue curve (m in equation 1) has been found for dense mixes to usually be between approximately 5 and 6 [28]. Softer grades of asphalt may however give steeper slopes. Base course mixes having lower asphalt contents and a softer grade asphalt tend to have a steeper slope than surface course mixes.

Brown and Pell [27] have developed a generalized nomograph for predicting fatigue life based on the results of extensive laboratory tests performed on a wide range of mixes. This nomograph which is given in Fig. 2 indicates that the most important factors influencing fatigue life of a mix are as follows: (1) tensile strain, (2) softening point temperature as defined by the ring and ball test and (3) the volume of binder.

Francken and Verstraeten [28] have developed a generalized equation for predicting fatigue life from tests performed on 42 mixes:

$$\varepsilon_r(N) = K' \, N_f^{-a} \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (2)$$

From their test results the slope of the fatigue curves was found to be almost the same, with the coefficient "a" in equation (2) being 0.21 (standard deviation 0.02). From the test data the following generalized expression was developed:

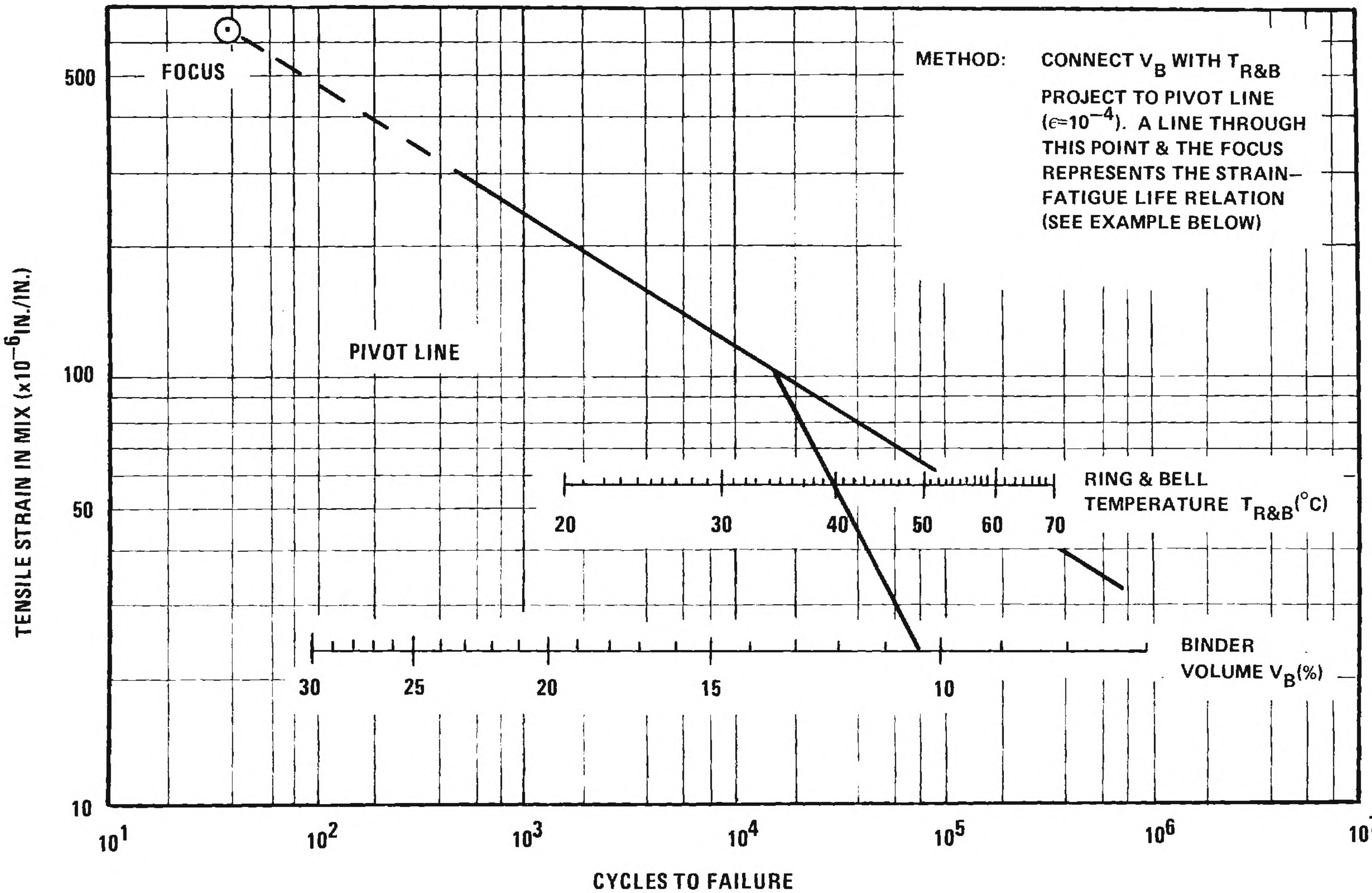


FIGURE 2. NOMOGRAPH FOR PREDICTING THE FATIGUE PERFORMANCE OF ASPHALT CONCRETE MIXES (AFTER BROWN AND PELL, REF. 45)

$$\varepsilon_r(N) = \lambda G \left[\frac{V_B}{V_B + V_v}\right] \left(\frac{N_f}{10^6}\right)^{-0.21} \quad \ldots\ldots\ldots \quad (3)$$

where $\varepsilon_r(N)$ = initial tensile strain causing failure

N_f = number of load repetitions required to cause failure applied at the initial strain level ε_r

λ = correction factor dependent upon the asphaltene content of the asphalt (or the ring and ball temperature). The correlation relationship for λ is given in Fig. 3

V_B = volume of bitumen (percent)

V_v = volume of voids present in the mix (percent)

G = this correlation factor is 1.0 when the volume of aggregate is between 78 and 85 percent of the total volume, and at least 50 percent of the aggregate is coarse. Therefore, for usual mixes the factor G would generally be one.

Equation (3) was developed from sinusoidal bending tests of the controlled stress type. Francken and Verstraeten [28] indicate that the effects of such factors as grading, nature and shape of aggregates, and the compaction method are indirectly considered through the factor $V_B/(V_B + V_v)$. They recommend that for optimum fatigue life the asphaltene content should be greater than 18 percent, and the ring and ball softening temperature should be greater than 118°F (48°C). This criteria is intended to insure that the bitumen used will not result in a mix having a fatigue life significantly lower than that obtained using a more suitable bitumen (refer to Fig. 3). Based on field observations, Francken and Verstraeten recommend that the penetration of the bitumen should be greater than 40, and the asphaltene content less than 27 percent.

Both the nomograph developed by Brown and Pell and the equation presented by Francken and Verstraeten were developed from bending tests using a sinusoidal loading. Therefore, these approaches may significantly underpredict the fatigue life that would exist when a rest period, similar to that which would occur in the pavement, occurs between each load pulse. Nevertheless, these simplified approaches can be used for preliminary comparisons of the relative fatigue life of various mixes.

Influence of Test Variables

A summary of the effect of some of the more important variables on fatigue life is given in Table 1. Pell and Cooper [5] have pointed out that the stiffness of the mix is

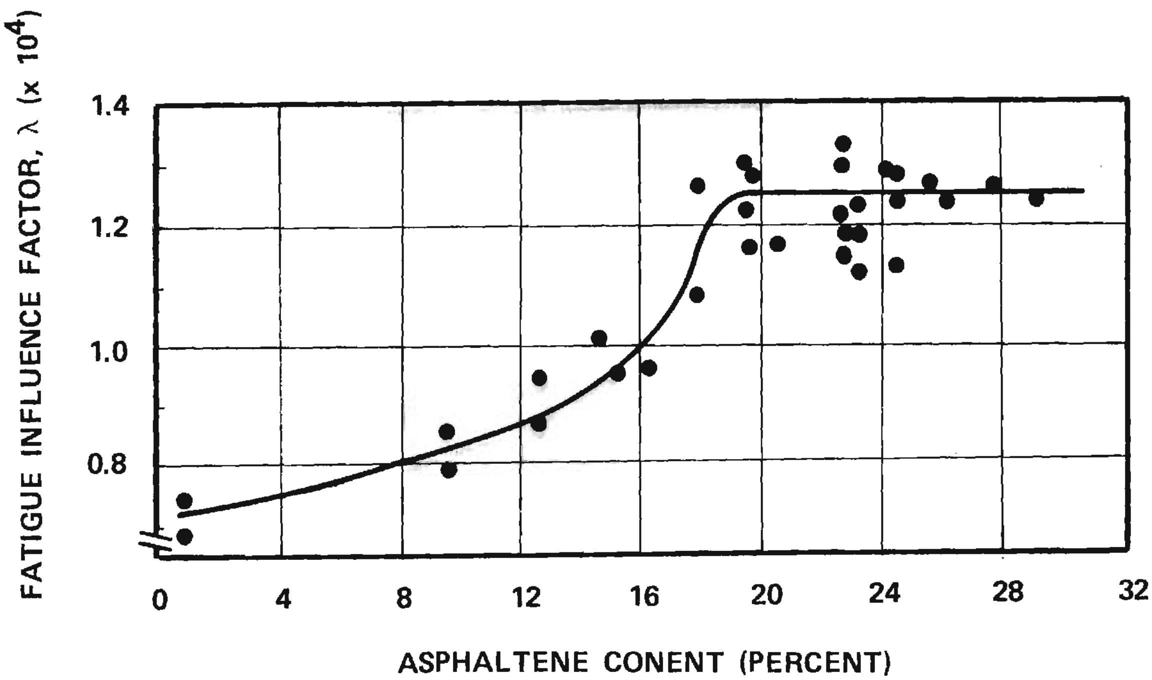


FIGURE 3. VARIATION OF λ AS A FUNCTION OF THE ASPHALTENE CONTENT OF THE BITUMEN (AFTER REF. 34)

TABLE 1. FACTORS AFFECTING THE STIFFNESS AND FATIGUE BEHAVIOR OF ASPHALT CONCRETE MIXTURES.

Factor	Change in Factor	Effect of Change in Factor		
		On Stiffness	On Fatigue Life	
			In Controlled Stress Mode	In Controlled Strain Mode
Asphalt Penetration	Decrease	Increase	Increase	Decrease
Asphalt Content	Increase	Increase[1]	Increase[1]	Increase[2]
Aggregate Type	Increase Roughness and Angularity	Increase	Increase	Decrease
Aggregate Gradation	Open to Dense Gradation	Increase	Increase	Decrease[4]
Air Void Content	Decrease	Increase	Increase	Increase[4]
Temperature	Decrease	Increase[3]	Increase	Decrease

[1] Reaches optimum at level above that required by stability considerations.

[2] No significant amount of data; conflicting conditions of increase in stiffness and reduction of strain in asphalt make this speculative.

[3] Approaches upper limit at temperature below freezing.

[4] No significant amount of data.

a dominant factor in determining fatigue behavior. As a result, for asphalt concrete thicknesses greater than about 6 in. (152 mm), a mix should be designed for maximum tensile stiffness and have the minimum practical amount of voids [26] as indicated by equation (3). Other variables influencing the stiffness of the mix include the binder characteristics (ring and ball softening temperatures and asphaltene content), degree of compaction, aggregate type, and grading. For practical mixes, aggregate gradation has been found to apparently have only a slight influence on fatigue performance. For relatively small changes in gradation, the volume of the binder is not significantly changed (usually the change in the volume of the binder is less than one percent). Therefore as indicated by equation (3), a small change in gradation would not be expected to have a significant effect on fatigue life. An optimum asphalt content has been found for at least some mixes beyond which the fatigue life decreases [22,25]. Moore and Kennedy [22] have found that higher compacting and mixing temperatures cause an increase in fatigue life.

Pulse shape has been found to be a reasonably important variable in fatigue testing [8]. Relative fatigue lives obtained from the direct tension test for a square, sinusoidal and triangular load pulse were found to be 0.42, 1.0 and 1.45, respectively. These results suggest that fatigue life may decrease with increasing energy input. Since the actual pulse developed in a pavement is close to a haversine shape, the fatigue life would probably be somewhere between that obtained using a sine and triangular wave pulse having the same duration. Kirk [11] has found that a large void in the specimen causes a significant reduction in fatigue life. As a result, in gap-graded mixes the ratio of fine to coarse aggregate should be relatively large to minimize the number of large voids. Further, Kirk found that the percent mineral filler should be at least equal to the asphalt content. Larger maximum size of aggregate was found to improve the fatigue properties of the mixes investigated.

Raithby and Ramshaw [18] found that traffic compaction in a large test slab increased the fatigue life for a given stress level by a factor of 3 and increased the dynamic stiffness by 60 percent. Traffic compaction was performed by a single constant wheel loading which moved up and down while at the same time progressing laterally across the surface of the test slab.

Direct tension fatigue tests were performed on specimens cut from the slab before and after traffic compaction. A direct comparison of the fatigue curves, when plotted in terms of strain, indicated that the specimens not compacted by traffic when subjected to the same strain level would have longer fatigue lives than specimens compacted by traffic. The explanation for this apparent discrepancy between observed fatigue life and that indicated by the fatigue curves is due primarily to the difference in stiffness of the two mixes. For mixes having different stiffnesses, the tensile strain in the asphalt concrete usually developed in the field would be different for equal loadings. As a result, the asphalt concrete compacted by traffic, for a given loading, would develop a lower tensile strain than the asphalt concrete not subjected to traffic. These test results clearly indicate an important concept that apparently in the past has not been given adequate consideration: *Fatigue curves when presented in terms of strain for at least certain mixes should not be directly compared to obtain an indication of relative fatigue performance.* This important concept will be considered further in Chapter 7.

Raithby and Ramshaw [18] also found that hydraulic oil accidentally spilled on a portion of the test pavement reduced the fatigue life by a factor of approximately six. This result further emphasizes the need for additional investigations of environmental factors such as road oils, weathering, temperature and moisture gradients on the fatigue life of asphalt concrete.

Rutting in Asphalt Concrete

Mechanism of Rutting

Rutting in asphalt concrete gradually develops with increasing wheel loadings, and often appears as a longitudinal depression in the wheel path. Rutting in asphalt concrete is caused by a combination of densification (decrease in volume) and shear deformation (plastic flow with no volume change). Trenching studies performed at the AASHO Road Test [30] and also the test track studies reported by Hofstra and Klomp [31] indicate that lateral (plastic) flow of the asphalt concrete is the primary rutting mechanism rather than densification. Further, the point of application of the wheel loading changes in the lateral direction with each vehicle. As a result, each successive loading will partially cancel out the lateral flow caused by the previous load applications.

Hofstra and Klomp found that the deformation through the asphalt concrete layer was greatest near the load and gradually decreased with depth below a certain level. Since rutting is caused by plastic flow, the distribution of rutting with depth observed by Hofstra and Klomp appears to be reasonable since more resistance to plastic flow should be encountered at greater depths beneath the wheel loading. Theoretical computations using layered theory and laboratory measured rutting properties performed by McLean and Monismith [32] and Morris et al. [35] indicated that most rutting should occur in the lower part of the asphalt concrete layer. The apparent discrepancy between the distribution of rutting with depth observed by Hofstra and Klomp, and that theoretically calculated by Morris et al. and McLean and Monismith indicates the need to establish by full-scale field rutting tests the actual distribution of rutting with depth through the asphalt concrete.

In relatively thin asphalt concrete pavements rutting occurs in not only the surfacing but also the base and subgrade. Measurements made at the AASHO Road Test [30] indicate that the surface rut depth reaches a limiting value for asphalt concrete thicknesses greater than approximately 10 in. (254 mm). Test track studies reported by Hofstra and Klomp [31] and also a theoretical study by McLean and Monismith [32] both indicate the existance of a threshold asphalt concrete thickness beyond which rutting does not increase. These results strongly indicate that for practical purposes all rutting is confined to the asphalt concrete layer for subgrades of reasonable strength when the thickness is greater than the threshold value.

The limiting allowable rut depth is controlled by both safety and structural considerations. Pavement failure in the United Kingdom is usually defined as a rut depth of 0.75 to 0.8 in. (19 - 20 mm) measured by a 6 ft. (1.8 m) straight edge [33]. Rut depths up to approximately 0.4 in. (10 mm) depth have been found not to cause any significant loss of structural strength. For a cross slope of 2.5 percent which is generally used in the United Kingdom, Lister and Addis [33] have found that depths greater than approximately 0.5 in. (13 mm) result in ponding of water for the 2.5 percent cross slope usually used which could cause hydroplaning or loss of skid resistance. Verstraeten et al [34] have found that the slope of the rut profile should not exceed 0.02 for good riding quality.

Prediction of Rutting

Several procedures have recently been proposed for predicting rutting in flexible pavements. In general these methods can be categorized as follows: (1) layer strain methods which use linear or nonlinear elastic layered theory and material properties measured from the repeated load triaxial test [1,32,35], (2) linear viscoelastic layered theory and material properties measured using creep tests or repeated load triaxial tests [36-39], and (3) simplified procedures using the results of creep tests [40-44]. The complexity of the proposed methods is a very important practical consideration in selecting one for use as a routine design procedure. The proposed methods vary in complexity from those requiring only a single creep test to very complex ones that require extensive laboratory tests and finite element analyses. Several of the more promising practical methods are reviewed with emphasis placed on their suitability for routine use by the Georgia Department of Transportation.

Barksdale and Leonards [36] proposed a relatively complicated approach using linear viscoelastic theory, a repeated stationary loading, and material properties obtained from the repeated load triaxial test. This method was found to give rut depths after 100,000 repetitions which were only 1.3 times larger than those measured at the AASHO Road Test. The approach, however, is too involved to be suitable for routine use. Elliot and Moavenzadeh [37] have developed a somewhat similar method using linear, viscoelastic theory and creep test results. In an early attempt to verify this approach Drennon and Kennis [38] found the observed rut depth to be three times the predicted one. The theory was subsequently modified, and new attempts to verify the method using rut depths measured in actual pavements indicate reasonable agreement with theory. Recently Battiato et al [39] have presented a linear viscoelastic method for predicting rut depth which uses a moving wheel loading and creep test results.

Barksdale [1] has proposed a simplified engineering theory for predicting rut depth using plastic material properties evaluated from the repeated load triaxial test together with either linear or nonlinear elastic layered theory. To predict the amount of rutting that would occur after a given number of load repetitions using the layer strain method, each layer of the pavement structure is divided into several fictitious sublayers, and the major principal stress and average confining stress is calculated at the center of each sublayer beneath the wheel load. Using the average stress state existing at the center of each sublayer, the corresponding plastic strain can be obtained from repeated load triaxial test results for the desired number of load repetitions. This general approach has been found by McLean and Monismith [32] and Morris et al [35] to give reasonably good predictions of the rut depth in asphalt concrete. The layer strain method offers a practical approach for predicting rutting in asphalt concrete provided that the average stress condition in the layer can be found so that only a single repeated load test has to be performed.

The Shell Laboratory, Amsterdam has conducted extensive studies in using the unconfined creep test to predict the rut depth in asphalt concrete [40-44]. From the results of a small scale track study, Uge and Van de Loo [41] found that the Marshall Mix

Design Method can be used to select the approximate optimum asphalt content for maximum resistance to rutting. Test track studies indicated that rut depth can more reliably be predicted using creep test results. Special parking tests conducted on the test track gave the next best prediction of rut depth, while the Marshall Mix Design Method was found to give the poorest comparisons with measured rut depths. In these comparisons, the creep tests were performed on cores taken from the test track whereas the Marshall tests were performed on laboratory prepared specimens.

To obtain a good comparison between relative rut depths observed in the test track and those calculated using the creep test results, Van de Loo [42] has found that the creep test must be performed at a relatively low stress level in the linear range of the materials. Van de Loo [40] has found that creep tests performed at a stress level of 15 psi (103 kN/m^2) give good results. The need for using a stress level in the linear range of the material has been attributed to the fact that the loading time in the field is small compared to the loading time in the creep test.

Van de Loo [42] has found that an asphalt concrete mix can be completely characterized by a plot of the stiffness of the mix as a function of the stiffness of the bitumen. Significant characteristics of this relationship are the slope and position of the curve. The stiffness of the mix is defined as the total strain in the creep test specimen divided by the stress and is time dependent (and hence depends upon the stiffness of the bitumen). The stiffness of the bitumen is a function of the properties of the binder, time of loading, and test temperature. The stiffness of the bitumen is usually obtained from the Van der Poel Nomograph [43], but can be obtained directly from laboratory tests. The method developed by the Shell Laboratory for estimating rut depth is summarized as follows [40]:

$$\Delta H = C_m H_o \left(\frac{\sigma_{avg}}{S_{mix}} \right) \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad (4)$$

where ΔH = Estimated rut depth in layer of thickness H_o

S_{mix} = Stiffness of the mix obtained from the creep curve at the value of the stiffness of the bitumen corresponding to the design life of the pavement

H_o = Layer thickness

C_m = Correction factor which includes dynamic effects. This factor depends on the type of mix and must be empirically determined

σ_{avg} = Average stress due to the moving wheel load in the layer in which rutting is being calculated. This quantity can be calculated using an appropriate layered system analysis.

To determine the stiffness of the bitumen appropriate for the design life, the following expression can be used:

$$S_{bit,\ visc} = \frac{3}{\sum_{T,t_o} \left[\frac{Nt_o}{\eta}\right]} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ . \ (5)$$

where: N = total number of wheel passes

t_o = loading time of one wheel pass

η = bitumen viscosity, defined as $\eta = \frac{1}{3} \lim_{t \to \infty} \left\{S_{bit} \cdot t\right\}$

The value of S_{bit} depends on the asphalt temperature, vehicle speed, and traffic using the pavement.

The method proposed by the Shell research group using a single creep test is a practical approach for predicting rutting in asphalt concrete. Further, this method has been found to give good relative comparisons of the rutting characteristics of a wide range of mixes as shown in Fig. 4. For the range of mixes investigated, the Shell creep test method has been found to predict rut depth within a factor of approximately two (Fig. 5). The effect of wheel loads having varying magnitude and lateral distribution can be readily handled using this approach [42].

Influence of Test Variables

Primary variables affecting rutting of asphalt concrete are viscous flow characteristics of the asphalt mix, the internal friction of the mineral skeleton, and the number of aggregate point contacts. The aggregate gradation determines the number of point contacts and also is an important factor influencing the internal friction of the mineral skeleton.

Hills [44] has found that the method of mixing and compacting specimens can have an important effect on the measured stiffness of the mix and hence upon the rutting properties. For the following methods of compaction, the stiffness of the mix was found to vary from higher to lower values by a factor of approximately ten: Gyratory, Marshall Method, static loading and rolling.

Brown and Pell [45] have found that for unconfined repeated load tests the vertical stress level has a large influence on the permanent strain occurring from stresses of 15 to 70 psi (103 to 483 kN/m^2). The effect of stress level tended to become more pronounced at a greater number of load repetitions. The effect of confining pressure on permanent strain was found to be much less than the effect of stress level. For the mix

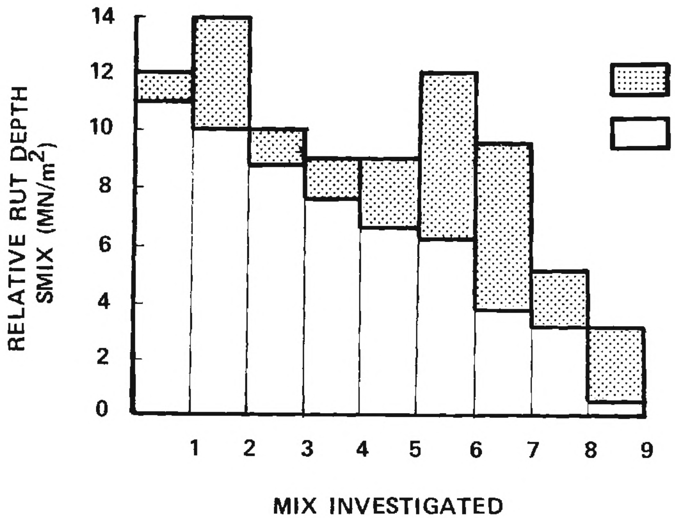


FIGURE 4. RELATIVE RUT DEPTHS MEASURED IN A LABORATORY TEST TRACK COMPARED WITH SHELL CREEP TEST RESULTS (AFTER VAN DE LOO, REF. 42)

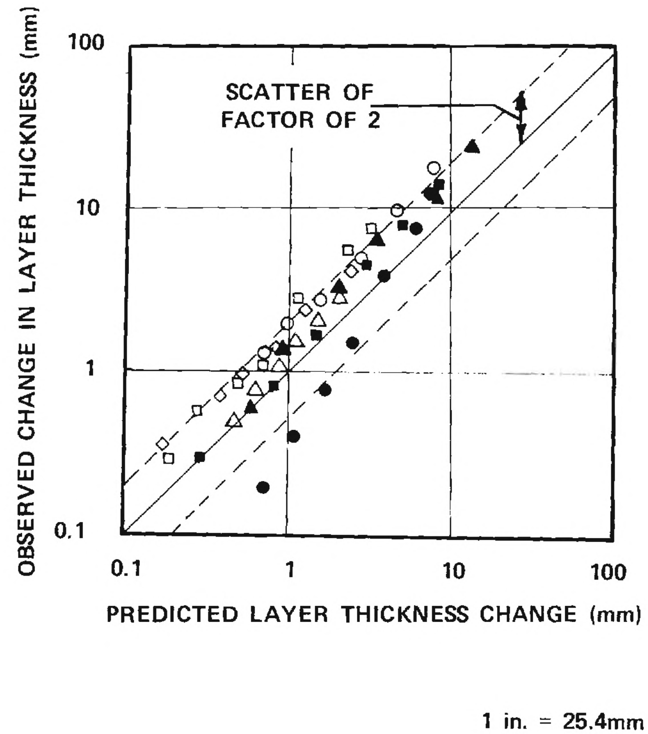


FIGURE 5. COMPARISON OF PREDICTED AND MEASURED CHANGE IN LAYER THICKNESS USING THE SHELL CREEP TEST METHOD (AFTER VAN DE LOO – REF. 42)

tested, Brown and Pell found that minimum permanent strain occurred for temperatures between 50 and 86°F (10 and 30°C) at an asphalt content of 4 percent with greater strains being observed at 3 and 5 percent binder contents. At a temperature of 104°F (40°C), however, 3 percent asphalt content was found to give the smallest permanent deformation.

Brown and Snaith [46] have found that a constant confining pressure equal to the average dynamic confining pressure can be used in performing repeated load triaxial tests. A rest period after the application of a load pulse was found to have little effect on permanent deformations. A longer pulse time did, however, give more deformation than faster pulses for the same total load pulse time.

Temperature has been found [1,31,35,45] to have a very significant effect on rutting. As shown in Fig. 6, Hofstra and Klomp [31] found from test track measurements that rutting increased by a factor of 250 to 300 going from a temperature of 68°F to 140°F (20 to 60°C).

Uge and Van de Loo [41] found that any changes in aggregate that increase the angle of internal friction of the mineral skeleton also tends to increase the stability of the mix. As the stability increases however, problems increase in placing and compacting the mix. McLean and Monismith [32] have found for relatively thick layers that the stiffness of the asphalt concrete has a significant influence on the amount of rutting occurring in the asphalt concrete layers, whereas stiffness of the subgrade has little effect.

Studies conducted by Brown and Pell [45] indicate that for the mix investigated, gap grading may give more deformation due to lower aggregate interlock than a continuously graded mix. Since aggregate interlock becomes more important at higher temperatures, gap graded mixes may be more susceptible to rutting at higher temperatures. The tendency for gap grades mixes to be more susceptible to rutting than continuously graded mixes tend to be confirmed by test track results [41]. A field study has found that a mix using a crushed river gravel is less susceptible to rutting than a similar one made of uncrushed river gravel [41]. Test track studies reported by Hofstra and Klomp indicate for the conditions investigated that rutting increases almost linearly with penetration grade and that a crushed sand asphalt is more stable than a sand asphalt having rounded grains.

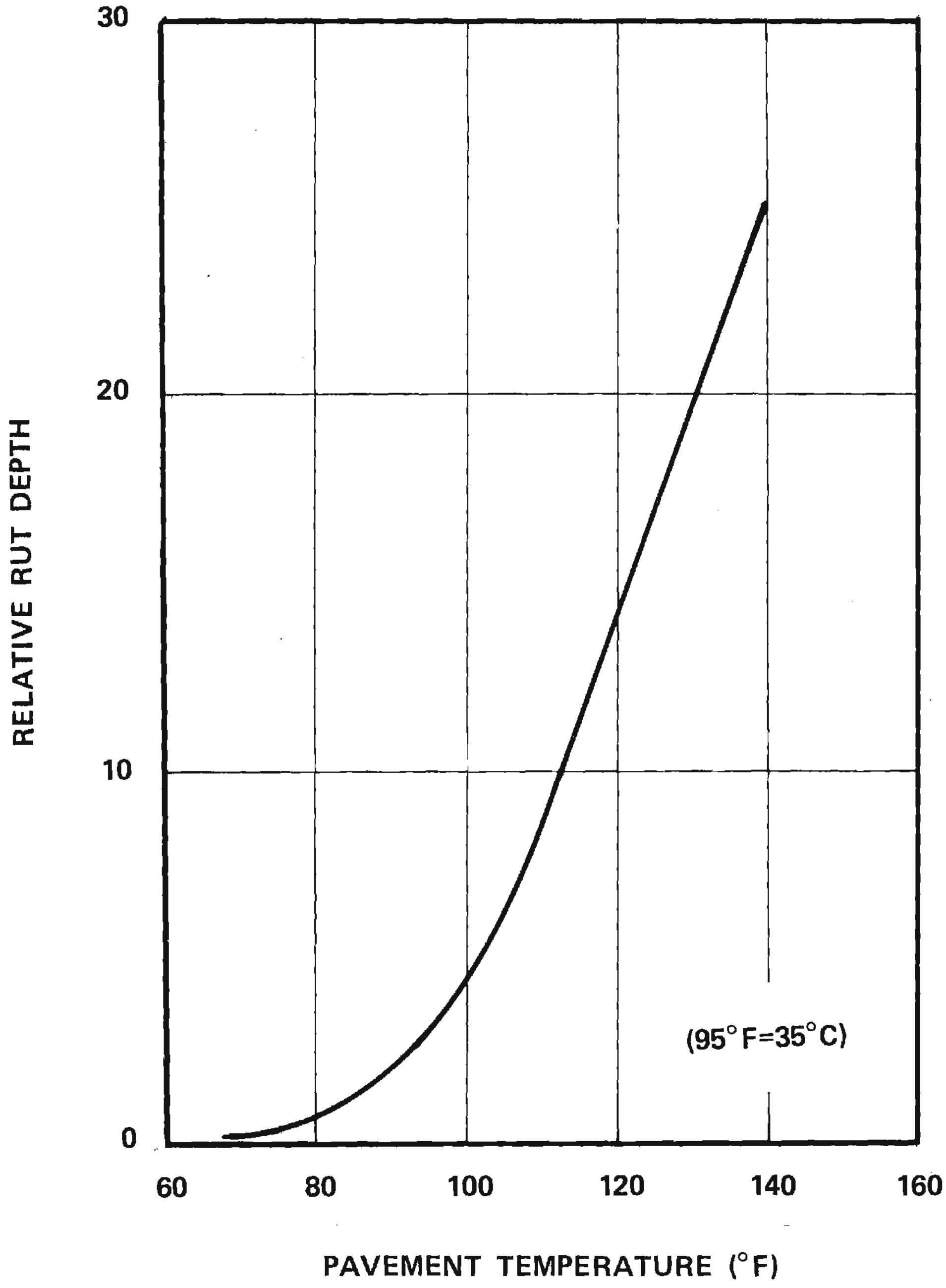


FIGURE 6. EFFECT OF PAVEMENT TEMPERATURE ON RUT DEPTH (AFTER HOFSTRA AND KLOMP, REF. 31)

CHAPTER 3

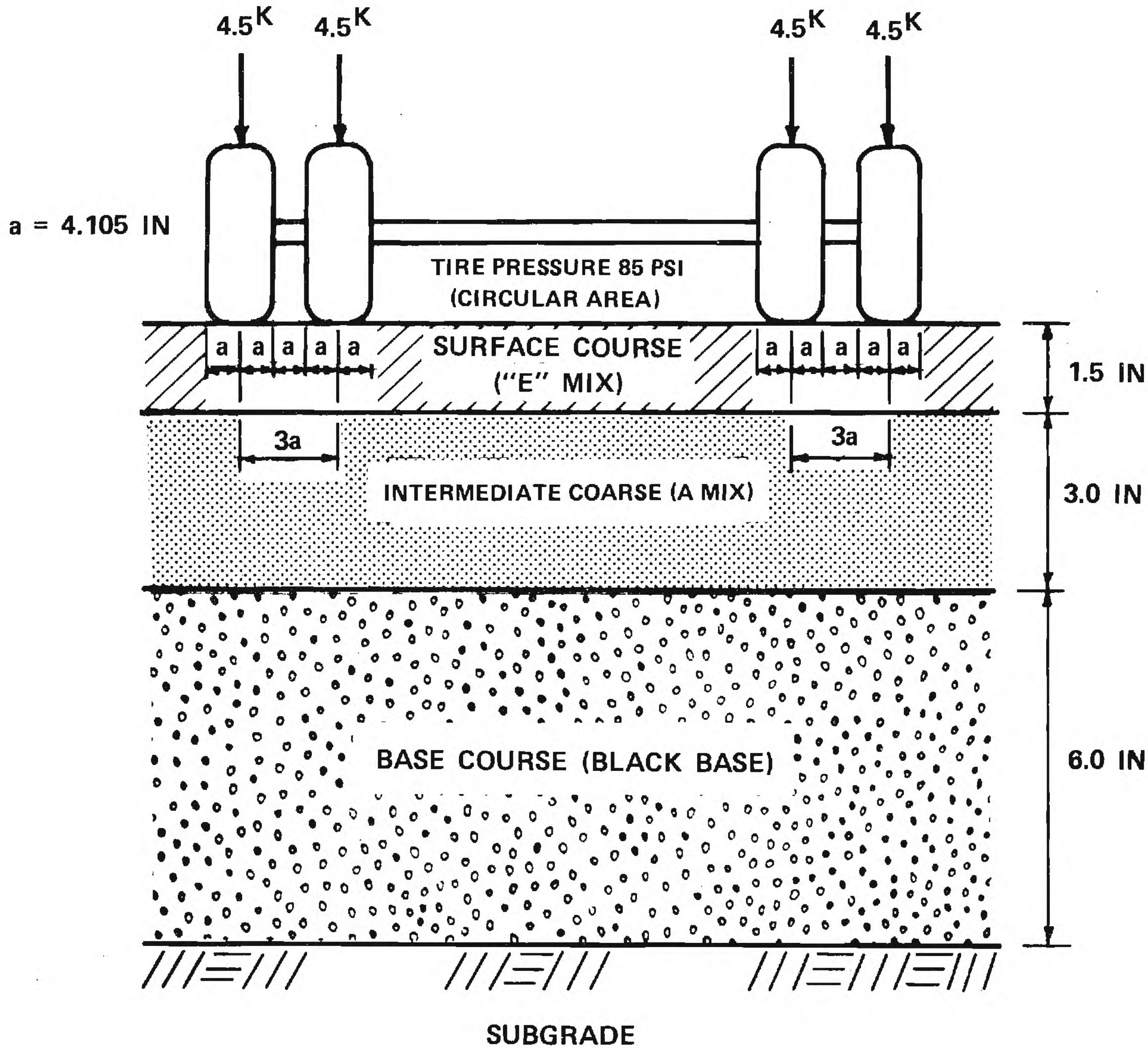
TEST CONDITIONS FOR LABORATORY EVALUATION OF ASPHALT CONCRETE MIXES

Introduction

Typical environmental and structural conditions existing in Georgia are analyzed in this chapter to determine suitable temperatures and stress states for simulating in the laboratory field conditions. A typical pavement section is assumed consisting of 10-1/2 in. (267 mm) full depth asphalt concrete layer such as one composed of a 6 in. (152 mm) black base, a 3 in. (76 mm) thick type "A" intermediate course, and a 1-1/2 in. (38 mm) type "E" surface mix (Fig. 7). This section is assumed to be placed directly on the subgrade. A standard 18-kip (80 kN) single axle load with dual wheels is considered to act on the structural pavement section. A tire contact pressure of 85.0 psi (586 kN/m^2) was used, and this loading was assumed to be uniformly distributed over a circular area. The contact area of the tire was calculated by dividing the gross load carried on the tire by the tire pressure giving a loading radius of 4.105 in. (104 mm).

The Chevron Five Layer Computer Program was utilized for calculating theoretical stresses and strains in the multi-layered flexible pavement section. Although asphalt concrete is actually nonlinear, a linearly elastic analysis was used as an engineering approximation. In the linear elastic layered theory used, each layer is characterized by an elastic modulus and Poisson's Ratio. For this analysis, Poisson's Ratio was taken to be 0.35 for the asphalt concrete layers and 0.40 for the underlying subgrade soil. The elastic modulus of the asphalt concrete is a function of the temperature, and the modulus of the subgrade was conservatively taken to be 4000 psi (27.6×10^3 kN/m^2) in the winter and 2000 psi (13.8×10^3 kN/m^2) in the spring and summer. The unit weight of the asphalt concrete was assumed to be 148 pcf (2.37 g/cc) and that of the subgrade 120 pcf (1.92 g/cc). All of these parameters were input to the Chevron CHEV5L computer program to analyze the stresses and deformations in the pavement. Superposition was then used to obtain the maximum stresses at a point in the pavement underneath the centerline of one wheel load due to a dual wheel loading.

A chart for use in selection of the duration of stress pulse time has been proposed by Barksdale [34] and is shown in Fig. 8. In this study, the stress pulse time was assumed to be approximately sinusoidal. For a constant vehicle speed of 45 mph (72.4 km/hr.), the duration of the stress pulse time at a depth of 6 in. (152 cm) below the surface of the pavement (i.e., at mid-depth of the combined intermediate and base



(1 in = 25.4mm ; 1 LBF = 4.45N)

FIGURE 7. APPLICATION OF AN 18–KIP SINGLE AXLE, DUAL WHEEL LOADING TO A TYPICAL ASPHALT CONCRETE PAVEMENT IN GEORGIA

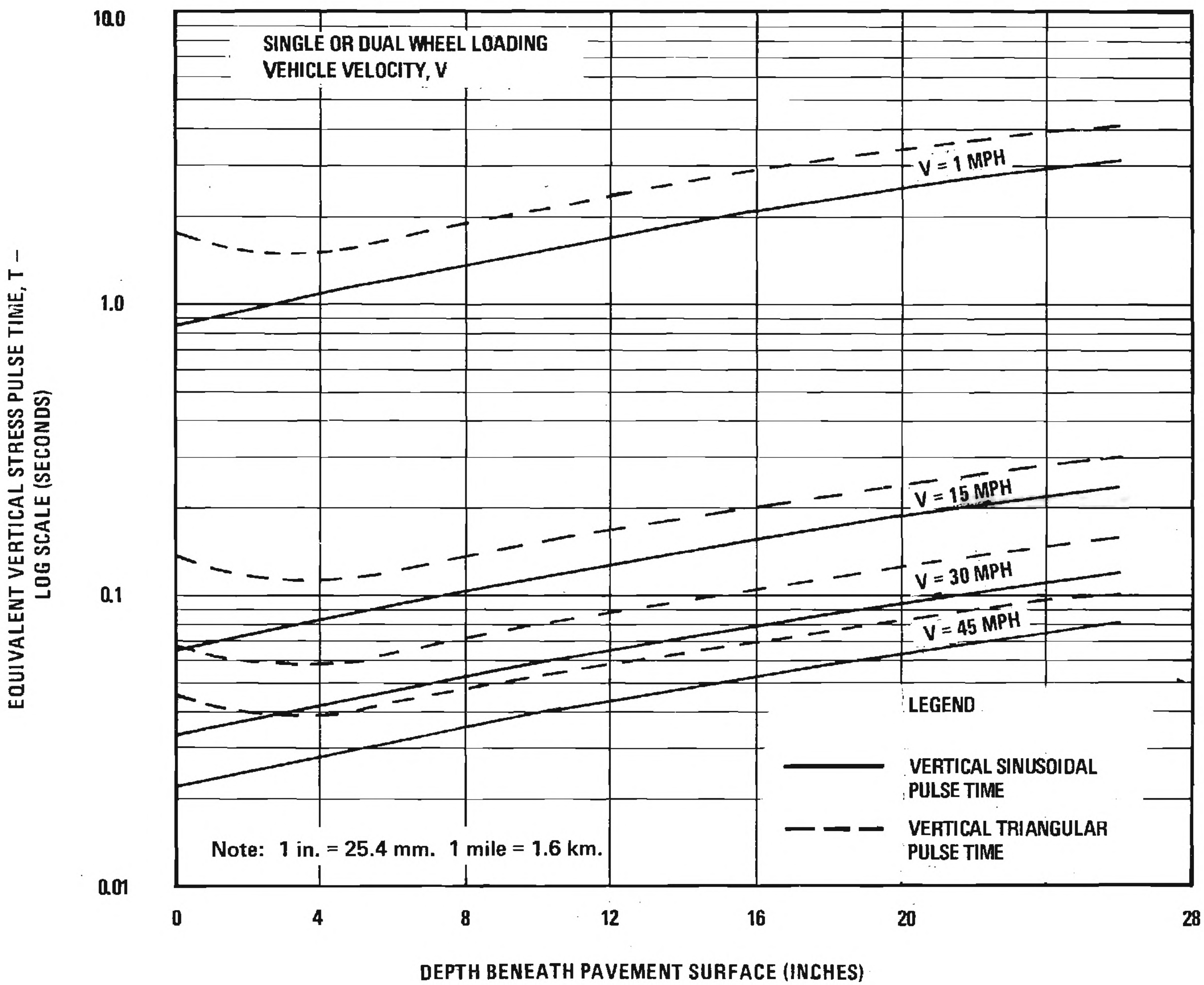


FIGURE 8. VARIATION OF EQUIVALENT VERTICAL STRESS PULSE TIME WITH VEHICLE VELOCITY AND DEPTH (AFTER REFERENCE 3)

courses) is approximately 0.032 sec. This loading time was used in selecting the temperature dependent modulus of elasticity of the asphalt concrete [47].

In-Situ Stress Conditions

The measurement of the permanent deformation characteristics of asphalt concrete specimens in the repeated load triaxial test requires that the specimens be tested under representative service conditions, which include a realistic stress state, temperature, and time of loading. The in-situ stresses were investigated using the results of the elastic analysis to determine appropriate values of vertical and horizontal stresses to be reproduced in the triaxial test.

An analysis [47] of rutting that considered effects of temperature variations indicated that half of the accumulated rutting in Georgia occurs at an average temperature in the asphalt concrete pavement above 95°F (35°C) [Fig. 9]. Therefore, 95°F (35°C) was chosen as a representative temperature for evaluating rutting characteristics in the laboratory using both the repeated load triaxial test and the creep test.

Elastic layered theory [47] indicated that the pavement structure could be realistically characterized by three repeated load triaxial tests: (1) One test in the compression zone, (2) one test at the neutral axis, and (3) one test in the tension zone. A constant confining stress was used in conjunction with a cyclic vertical stress. For theoretical purposes, a constant confining pressure was used to equal to two-thirds of the maximum horizontal stress. Fig. 10 summarizes the theoretical stress conditions that should be reproduced in the triaxial cell for the warmest and coldest seasons of the year. One shortcoming of the triaxial test is in handling the stress in the lower part of the asphalt concrete layer where horizontal tensile stress develops in both the lateral and horizontal directions.

Average Stress State - The Z Function

Only an axial tensile stress can be applied to the asphalt concrete specimens in the laboratory triaxial test. A considerably more practical alternative approach than reproduction of the stress states in the top, middle and bottom of the layer is a determination of the single "*average*" stress condition in the layer which should occur near the neutral axis in the compression zone. The determination of the appropriate *average* stress state using the Z Function approach is discussed in this section.

The two laboratory testing methods presently used in the evaluation of the pavement deformation characteristics of asphalt concrete are the repeated load triaxial test and

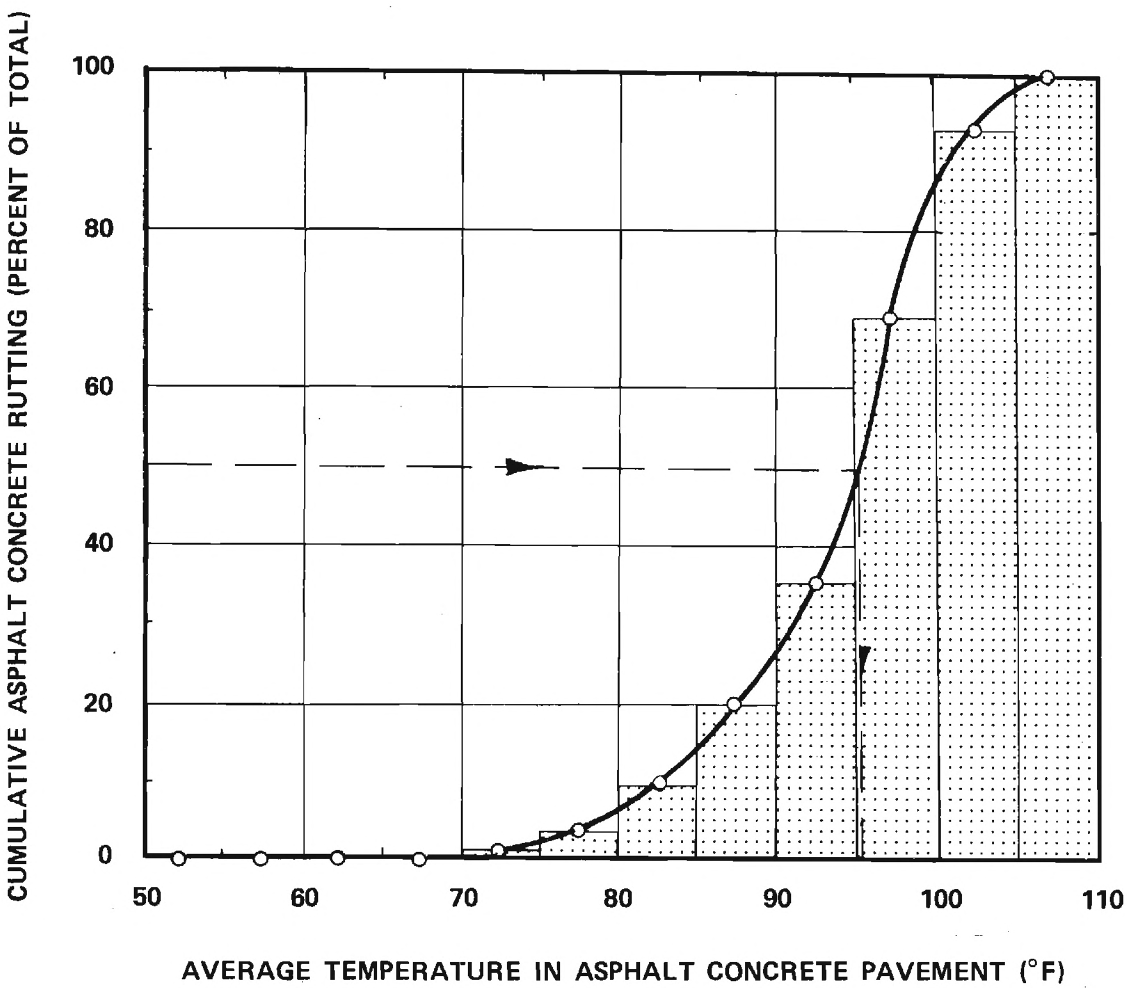
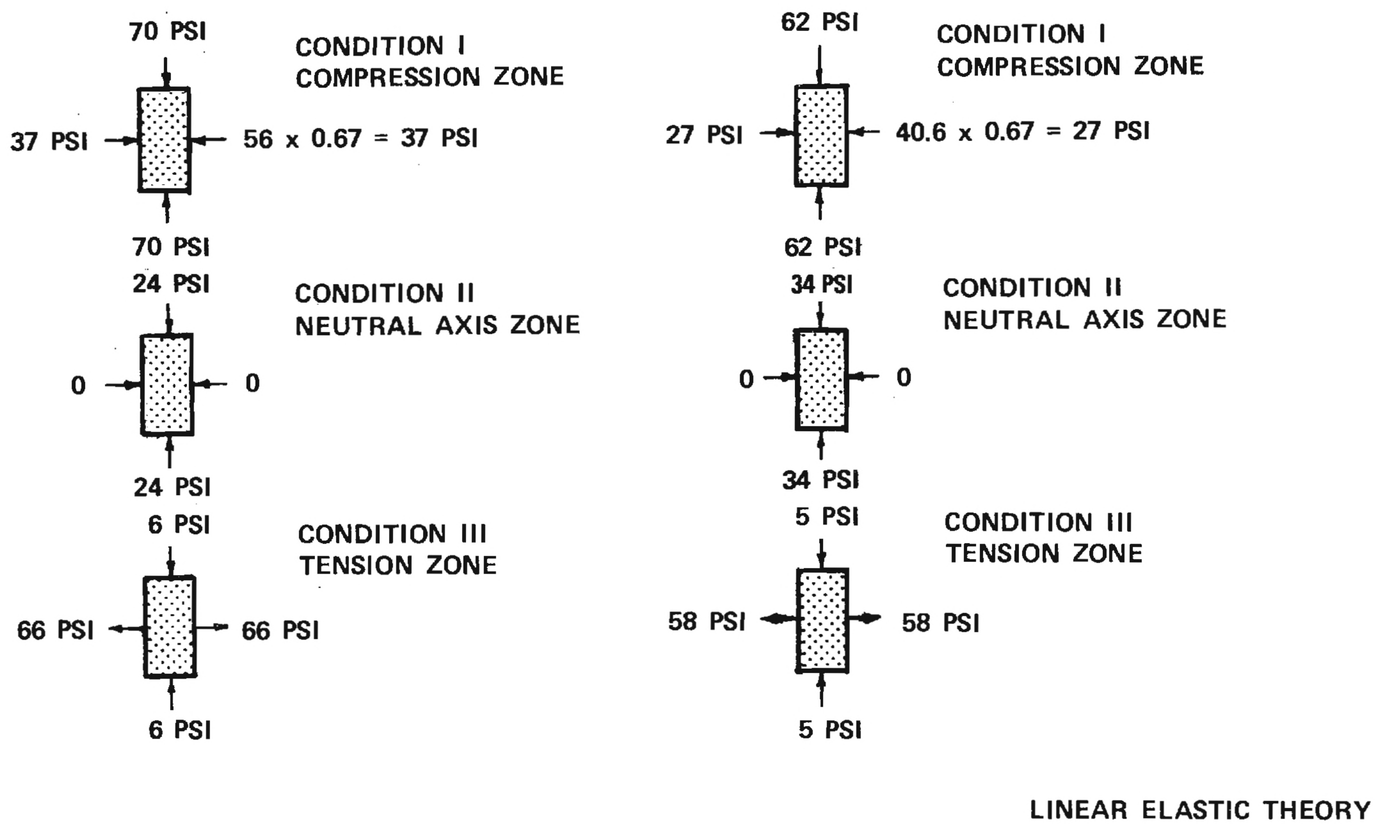


FIGURE 9. PAVEMENT TEMPERATURE AS A FUNCTION OF CUMULATIVE PERCENT OF ASPHALT CONCRETE RUTTING IN GEORGIA — TYPICAL CONDITIONS (AFTER REF. 47)



(a) SUMMER MONTHS

(b) WINTER MONTHS

FIGURE 10. AVERAGE THEORETICAL STRESS STATES FOR 10.5 INCH THICK FULL DEPTH PAVEMENT—18 KIP DUAL WHEEL LOADING (AFTER REF. 47)

the creep test. These two methods are quite different with respect to the method of stress application to the specimens. In the repeated load triaxial test, the in-situ principal stresses occurring at any depth in the asphalt concrete pavement are reproduced and a cyclic principal stress is applied. In the Shell creep test, the specimen is subjected to a constant compressive load and the time-deformation response is measured.

The unconfined creep test should be performed at the average axial stress state which will be representative of the rutting characteristics throughout the layer. This effective axial stress would be expected to be not too different from that occurring in the vicinity of the neutral axis. The stress applied to the specimen in the creep test should therefore be the average stress equivalent to the three-dimensional stress conditions occurring throughout the entire asphalt concrete layer depth. The theoretical prediction of average stress in any asphalt concrete layer can be made by using the Z function [40,44], which can be determined for convenience from an elastic layer program such as CHEV5L. Even though the theoretical prediction of average stress in the asphalt concrete layer is based on the elastic deformation of the layer, it does consider the combined effect of tension and compression in a three-dimensional stress field. Hence, this approach should give a reasonably representative stress state for use in testing to evaluate plastic deformation.

The Z function, which is dependent upon the pavement structure, geometry and wheel load spectrum, is defined as the ratio of the average vertical elastic strain occurring in the asphalt concrete layer to the vertical strain measured in the unconfined compression test of an asphalt concrete specimen. The Z function can then be calculated from elastic theory as follows:

$$Z_i = \left(\frac{\Delta H_i}{H_i}\right) \Big/ \left(\frac{\sigma_o}{E_i}\right) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (6)$$

where Z_i = Z function for the i^{th} layer

H_i = initial thickness in the i^{th} layer

ΔH_i = vertical displacement within the i^{th} layer evaluated from elastic theory

E_i = modulus of elasticity of the i^{th} layer

σ_o = tire contact stress at the surface of the asphalt concrete pavement

The average stress to apply to an asphalt concrete specimen in an unconfined creep test is derived by rearranging equation (6) as follows:

$$\frac{\Delta H}{H} = Z \cdot \left[\frac{\sigma_o}{E}\right] \quad . \; . \; . \; . \; . \; . \; . \; . \; . \qquad (7)$$

where $\Delta H/H$ is the average strain in the layer

$$\varepsilon_{layer} = Z\ (\sigma_o/E) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \qquad (8)$$

The average unconfined axial stress, σ_{avg} required to cause the same strain in the layer being considered is equal to

$$\varepsilon_{layer} = \sigma_{avg}/E \quad . \; . \; . \; . \; . \; . \; . \; . \; . \qquad (9)$$

Substituting equation (9) into equation (8) gives

$$\frac{\sigma_{avg}}{E} = Z\ (\sigma_o/E) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \qquad (10)$$

and upon rearranging equation (10)

$$\sigma_{avg} = Z\ \sigma_o \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \; . \qquad (11)$$

Equation (11) gives the vertical stress that would have to be applied to an unconfined specimen to give the same elastic strain as that calculated in the layer. The modulus of elasticity of the specimen and layer would be the same.

In this study the Z function for the surface and base course layers of two selected asphalt concrete pavements were evaluated for the 9-kip dual wheel loading. The results of this analysis for both winter, spring and summer environmental conditions are shown in Tables 2 and 3 for the structural pavement conditions shown in Fig. 7. For a similar loading applied to a 4.5 in. (114 mm) thick asphalt concrete layer placed over 6 in. (152 mm) of crushed stone, the Z function for the two layers are given in Table 4 for winter and summer environmental conditions. Of interest is the fact that the use of the theoretically calculated Z functions in equation (11) gives average axial stresses for summer temperatures which are higher than the vertical stress calculated at the neutral from elastic theory for summer conditions. These stresses, however, are quite close to the vertical stress calculated at the neutral axis for winter environmental conditions. Use of the Z functions give a preliminary indication of the *average* stress state within a layer for use in testing. Specific stress conditions, however, must be determined by correlating laboratory test results with measured values of rut depth from actual pavements.

TABLE 2. FUNCTION Z_1 FOR UPPER 4.5 IN. OF 10.5 IN. THICK FULL-DEPTH ASPHALT CONCRETE PAVEMENT LOCATED IN GEORGIA-DUAL WHEEL LOADING.

a/H_1	E_2/E_1	Z_1	E_1 (PSI)	SEASON
0.912	0.89	0.22	1,450,000	WINTER
	1.03	0.21	1,450,000	
	1.30	0.25	1,150,000	
	1.64	0.26	870,000	
	1.15	0.23	1,100,000	
	0.88	0.24	510,000	SPRING
	1.08	0.21	510,000	
	1.89	0.31	290,000	
	3.47	0.37	150,000	
	1.54	0.22	260,000	
	1.00	0.23	330,000	SUMMER
	1.15	0.23	330,000	
	2.00	0.32	190,000	
	3.85	0.39	96,000	
	1.71	0.22	170,000	

NOTE: Refer to Figure 7 for structural pavement section.

TABLE 3. FUNCTION Z_2 FOR LOWER 6.0 IN. OF 10.5 IN. THICK FULL-DEPTH ASPHALT CONCRETE PAVEMENT LOCATED IN GEORGIA - DUAL WHEEL LOADING.

a/H_2	E_3/E_2	Z_2	E_2 (PSI)	SEASON
0.684	3.07×10^{-3}	0.56	1,300,000	WINTER
	2.68×10^{-3}	0.57	1,490,000	
	2.68×10^{-3}	0.56	1,490,000	
	2.79×10^{-3}	0.53	1,430,000	
	3.14×10^{-3}	0.54	1,270,000	
	4.44×10^{-3}	0.49	450,000	SPRING
	3.64×10^{-3}	0.54	550,000	
	3.64×10^{-3}	0.54	520,000	
	3.85×10^{-3}	0.52	520,000	
	5.00×10^{-3}	0.50	400,000	
	6.06×10^{-3}	0.48	330,000	SUMMER
	5.26×10^{-3}	0.49	380,000	
	5.26×10^{-3}	0.51	380,000	
	5.40×10^{-3}	0.49	370,000	
	6.89×10^{-3}	0.48	290,000	

NOTE: Refer to Figure 7 for structural pavement section.

TABLE 4. FUNCTION Z_1 and Z_2 FOR A 4.5 IN. ASPHALT CONCRETE SURFACING OVERLYING A 6 IN. GRANULAR BASE LOCATED IN GEORGIA - DUAL WHEEL LOADING.

Z_1	E_2/E_1	a/H_1	E_1 (PSI)	SEASON	Z_2	E_3/E_2	a/H_2
0.38	2.41×10^{-2}	0.912	1,450,000	WINTER	0.12	1.14×10^{-1}	0.684
0.36	3.04×10^{-2}		1,150,000		0.13		
0.39	3.18×10^{-2}		1,100,000		0.15		
0.35	4.02×10^{-2}		870,000		0.13		
0.34	1.06×10^{-1}		330,000	SUMMER	0.24	5.71×10^{-2}	
0.34	1.84×10^{-1}		190,000		0.29		
0.37	2.06×10^{-1}		170,000		0.34		
0.35	3.65×10^{-1}		96,000		0.30		

NOTE: Function Z_1 for the asphalt concrete layer and Z_2 for the granular layer.

CHAPTER 4

MATERIAL PROPERTIES AND SAMPLE PREPARATION

Introduction

Asphalt concrete base course mixes having a wide range of material types and aggregate gradations were tested in this study. An AC-20 viscosity grade asphalt cement was used in most of the tests, although an AC-40 was used in one of the supplementary studies. Both 50 and 75 blow Marshall mixes were investigated for varying asphalt contents. Aggregates tested included granitic gneiss, limestone and a blend of crushed stone and hydraulic fill sand. The effect was also investigated of using flyash and portland cement filler instead of granitic gneiss mineral filler.

Material Properties

Crushed granitic gneiss and crushed limestone aggregate were used in the asphalt concrete base course mixes. The granitic gneiss and limestone aggregates were obtained from the Vulcan Material Company's Norcross Quarry and the Dalton Rock Product's Dalton Quarry, respectively. The physical properties of these crushed stone aggregates are summarized in Table 5. The aggregate gradations used in all mixes studied in this investigation fell within the Georgia DOT allowable specification limits for black base and modified B mixes.

The AC-20 viscosity grade asphalt cement used as the basic binder was obtained from the Shell Oil Company (Trumbull Products, Atlanta, Georgia). The AC-40 asphalt cement used in the supplementary tests to study the effect of asphalt viscosity was obtained from the Hunt Oil Company, Tuscaloosa, Alabama. The AC-20 viscosity grade asphalt cement used in the I-95 base course specimens was obtained from the American Oil Company (Savannah, Georgia). The physical properties of these asphalt cements are summarized in Table 6.

Marshall Mix Designs for each asphalt concrete mix studied in this investigation were performed by the Georgia Department of Transportation, Office of Materials and Research. A summary of the mixes investigated are given in Table 7 including viscosity grade asphalt cement, number of blows used in the Marshall Mix Design (50 or 75), gradation, and aggregate type. Table 7 serves as a reference guide to the detailed tabulation of data for each mix design given in Figs. 11 through 18. The figure number given in parentheses in the second column of Table 7 indicates the location of the

TABLE 5. PHYSICAL PROPERTIES OF THE CRUSHED STONE AGGREGATE USED IN THE ASPHALT CONCRETE MIXES.

PHYSICAL PROPERTY	MATERIAL	
	Vulcan Materials Co. (Norcross, Ga.)	Dalton Rock Prod. (Dalton, Ga.)
Aggregate Description	Granitic Gneiss	Limestone
Georgia DOT Class Aggr.	B	A
Specific Gravity		
Bulk	2.68	2.68
Apparent	2.72	2.76
S.S.D.	2.70	2.71
Absorption (%)	0.26	1.05
Sand Equivalent	85	91
L. A. Abrasion (%)	56	19
Mag. Sulfate Soundness Loss (%)	1.0	5.67

TABLE 6. SUMMARY OF THE ASPHALT CEMENT PROPERTIES USED IN BASE COURSE MIXES.

PHYSICAL PROPERTY	ASPHALT CEMENT		
	AC-20	AC-40	AC-20(I-95)
KINEMATIC VISCOSITY 140°F (Poises) 140°F (Poises) 275°F (Centistokes)	 1761 4072 358	 4833 653	 2341 4743 374
PENETRATION: 100 gm., 5 sec. (1/10 mm), 77°F	68	47	70
RING & BALL SOFTENING POINT ASTM (°F)	124	130	119
SPECIFIC GRAVITY	1.017	1.046	1.033
CLEVELAND FLASH POINT (°F)	590	575	550
DUCTILITY(2) 77°F (cm)	150+	150+	150+
SOURCE	Shell Oil Co. (Trumbull, Atl., Ga.)	Hunt Oil Co. (Tuscaloosa, Ala.)	American Oil Co. (Savannah, Ga.)

1. Kinematic viscosity of thin film residue.
2. Test performed on thin film residue.

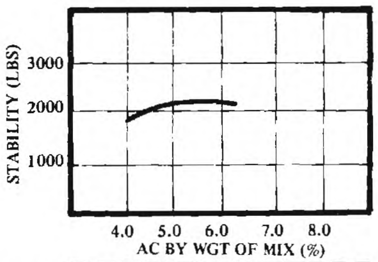
TABLE 7. SUMMARY OF MIX DESIGNS USED IN STUDYING FATIGUE AND RUTTING OF ASPHALT CONCRETE BASE COURSE MIXES.

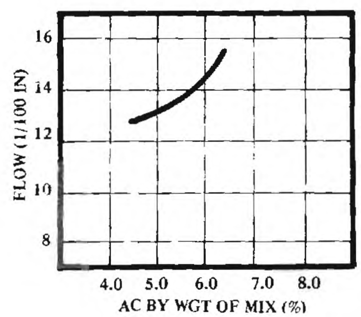
MIX NO.	DESIGNATION	MARSHALL DESIGN	ASPHALT CEMENT	AGGREGATE		
				TYPE	GRADATION	SOURCE
I. BLACK BASE MIXES						
1	N-BB (Fig. 11)	50 blow	AC-20 (Shell Oil)	Granitic Gneiss	A (Medium)	Norcross
2	N-75 (Fig. 12)	75 blow	AC-20 (Shell Oil)	Granitic Gneiss	A (Medium)	Norcross
3	N-BB-40 (Fig. 13)	50 blow	AC-40 (Hunt Oil)	Granitic Gneiss	A (Medium)	Norcross
4	NF-BB (Fig. 14)	50 blow	AC-20 (Shell Oil)	Granitic Gneiss	A (Medium)	Norcross
5	NC-BB (Fig. 15)	50 blow	AC-20 (Shell Oil)	Granitic Gneiss	C (Coarse)	Norcross
6	D-BB (Fig. 16)	50 blow	AC-20 (Shell Oil)	Limestone	A (Medium)	Dalton
II. MODIFIED B BASE						
7	NM-50 (Fig. 17)	50 blow	AC-20 (Shell Oil)	Granitic Gneiss	D (Fine)	Norcross
8	NM-75 (Fig. 18)	75 blow	AC-20 (Shell Oil)	Granitic Gneiss	D (Fine)	Norcross

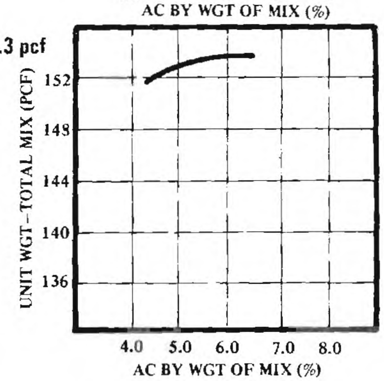
FIGURE 11. MARSHALL MIX DESIGN RESULTS FOR MIX 1-50 BLOW BLACK BASE

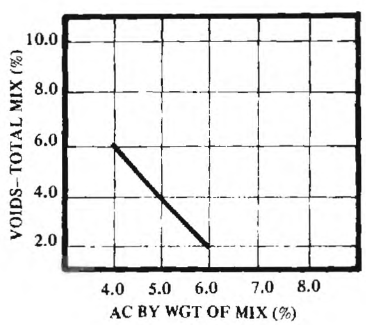
ASPHALT/SOURCE: AC-20/SHELL OIL CO. (ATLANTA)
R&B SOFT. PT.: 124 ° F
PEN. @ 77°F : 68
FILM THICKNESS: 9.5 MICRONS
MIXING TEMP : 285-295°F
COMPACTING TEMP : 262-278° F

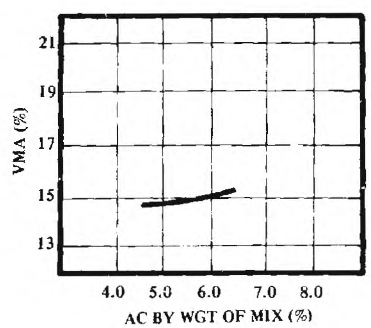
OPTIMUM % AC: 4.8
STABILITY (LBS.) : 2150
FLOW (1/100 IN): 12.8
% VOIDS-TOTAL MIX: 4.5
% VMA : 14.9
UNIT WGT-TOTAL MIX: 152.3 pcf











STONE/SOURCE: GRANITIC-GNEISS / NORCROSS
50 BLOW MARSHALL MIX DESIGN
TYPE OF MIX: BLACK BASE

AGGREGATE GRADATION

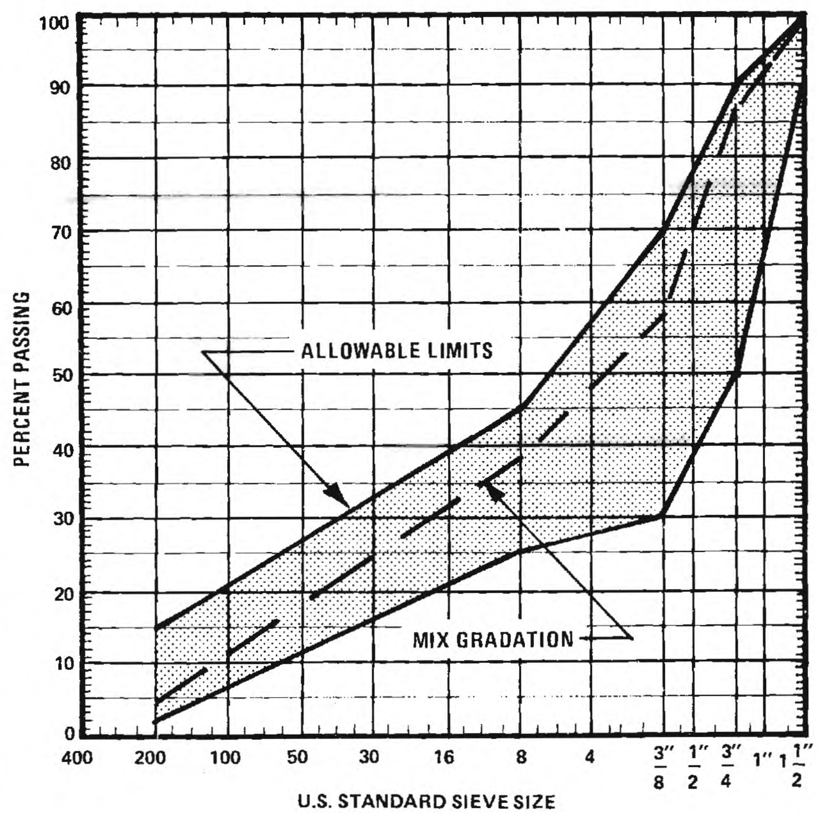
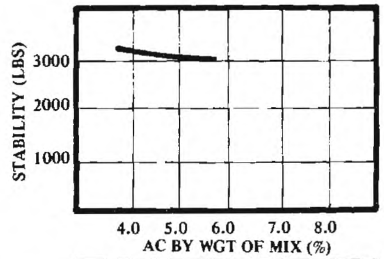


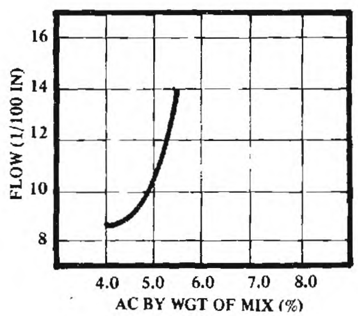
FIGURE 12. MARSHALL MIX DESIGN RESULTS FOR MIX 2–75 BLOW BLACK BASE

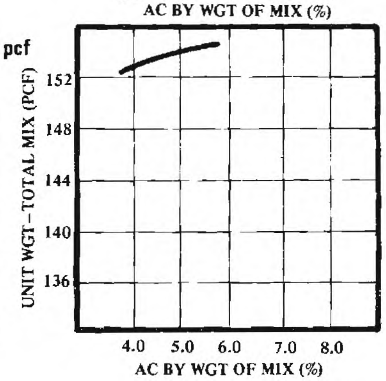
ASPHALT/SOURCE: AC–20 / SHELL OIL CO. (ATLANTA)
R&B SOFT. PT.: 124 °F
PEN. @ 77°F : 68
FILM THICKNESS: 9.5 MICRONS
MIXING TEMP : 285–295° F
COMPACTING TEMP : 262–278 °F

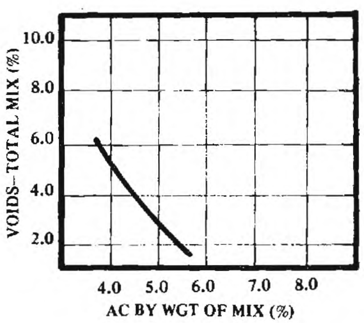
OPTIMUM % AC: 4.5
STABILITY (LBS.): 2152
FLOW (1/100 IN): 8.0
% VOIDS–TOTAL MIX: 3.8
% VMA : 14.3
UNIT WGT–TOTAL MIX: 152.9 pcf

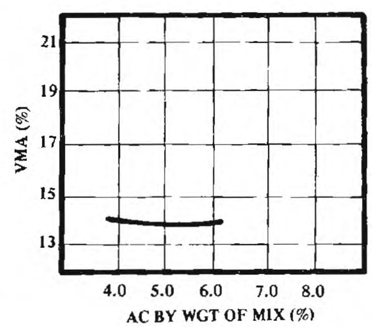
STONE/SOURCE: GRANITIC GNEISS / NORCROSS
75 BLOW MARSHALL MIX DESIGN
TYPE OF MIX: GEORGIA BLACK BASE











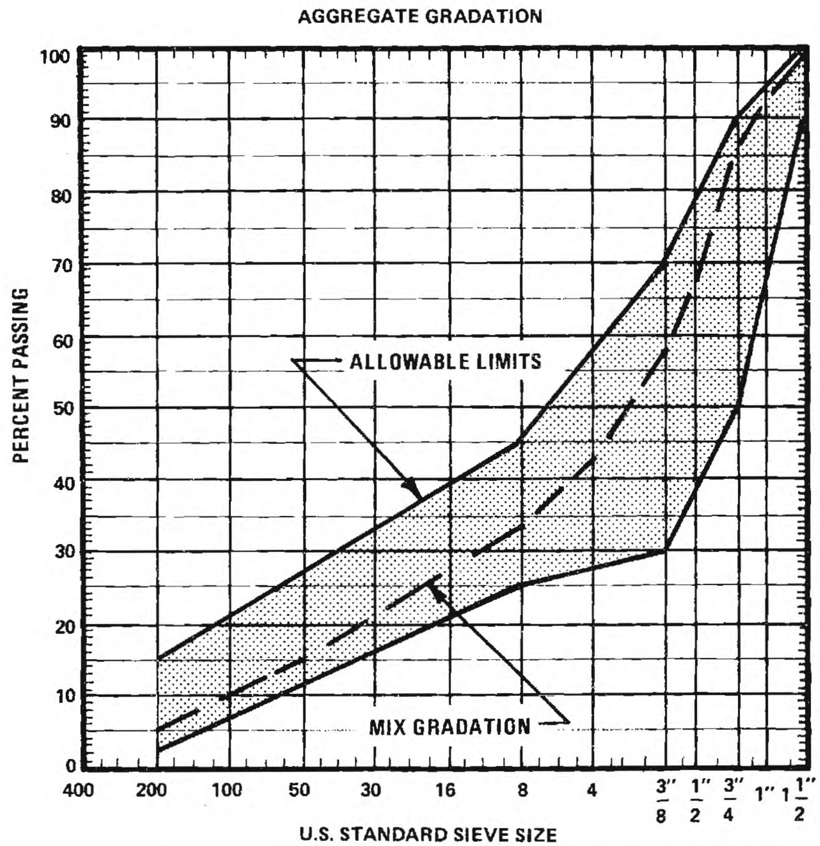
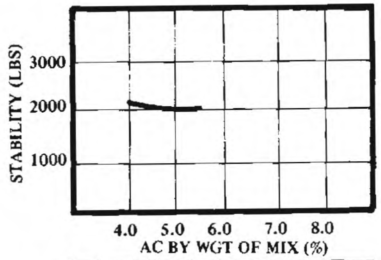


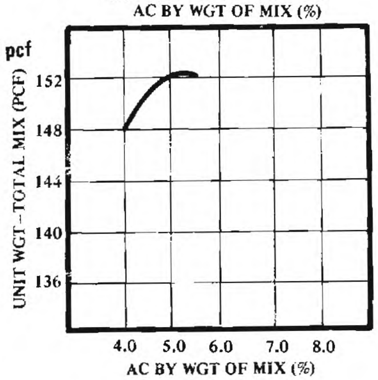
FIGURE 13. MARSHALL MIX DESIGN RESULTS FOR MIX 3–BLACK BASE WITH AC–40

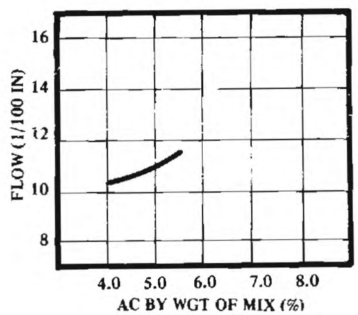
ASPHALT/SOURCE: AC–40/SHELL OIL CO. (ATLANTA)
R&B SOFT. PT.: 130° F
PEN. @ 77°F : 47
FILM THICKNESS: 9.9 MICRONS
MIXING TEMP : 328–342° F
COMPACTING TEMP : 305–315 ° F

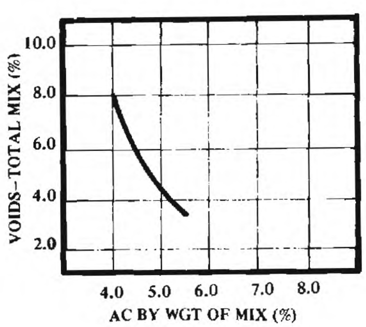
OPTIMUM % AC: 5.0
STABILITY (LBS.) : 2000
FLOW (1/100 IN): 11.0
% VOIDS–TOTAL MIX: 4.5
% VMA : 15.4
UNIT WGT–TOTAL MIX: 151.8 pcf

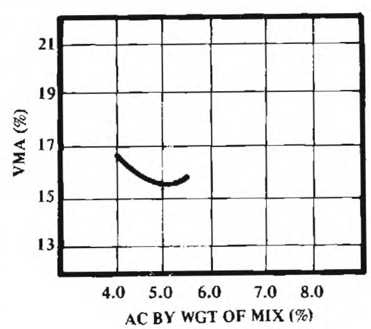
STONE/SOURCE: GRANITIC–GNEISS
50 BLOW MARSHALL MIX DESIGN
TYPE OF MIX: BLACK BASE











AGGREGATE GRADATION

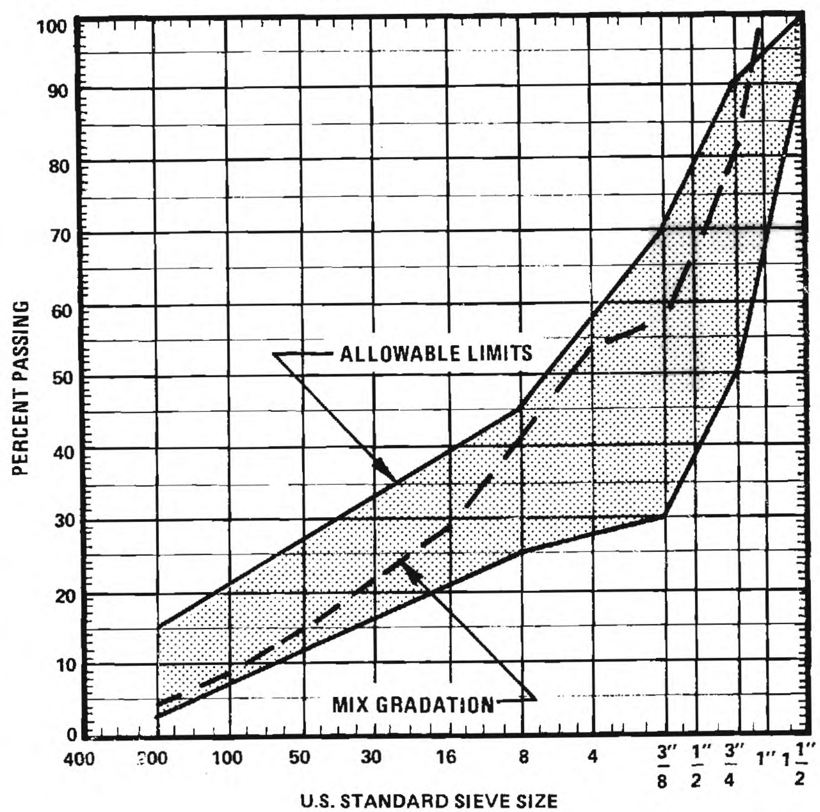


FIGURE 14. MARSHALL MIX DESIGN RESULTS FOR MIX 4-50 BLOW BLACK BASE WITH FINE GRADATION

ASPHALT/SOURCE: AC-20 / SHELL OIL CO. (ATLANTA)

R&B SOFT. PT.: 124 °F

PEN. @ 77°F : 68

FILM THICKNESS:

MIXING TEMP : 285-295 °F

COMPACTING TEMP : 262-278° F

OPTIMUM % AC: 4.5

STABILITY (LBS.) : 1716

FLOW (1/100 IN): 9.0

% VOIDS-TOTAL MIX: 6.8

% VMA : 16.4

UNIT WGT-TOTAL MIX: 149.2 pcf

STONE/SOURCE: GRANITIC GNEISS / NORCROSS

50 BLOW MARSHALL MIX DESIGN

TYPE OF MIX: BLACK BASE

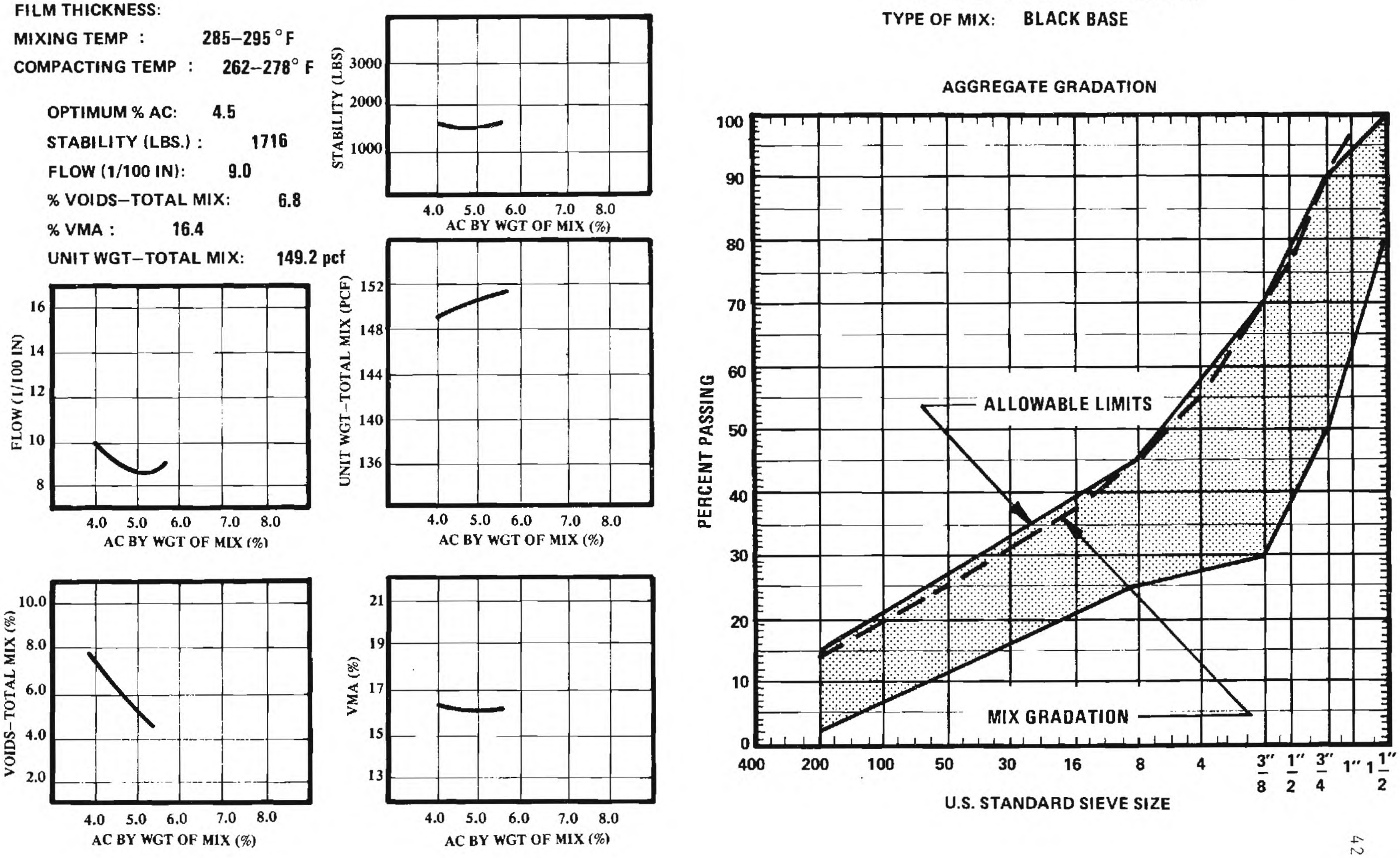


FIGURE 15. MARSHALL MIX DESIGN RESULTS FOR MIX 5–50 BLOW BLACK BASE WITH COARSE GRADATION

ASPHALT/SOURCE: AC–20 / SHELL OIL CO. (ATLANTA)

R&B SOFT. PT.: 124° F

PEN. @ 77°F : 68

FILM THICKNESS:

MIXING TEMP : 285–295° F

COMPACTING TEMP : 262–278 ° F

OPTIMUM % AC: 4.5

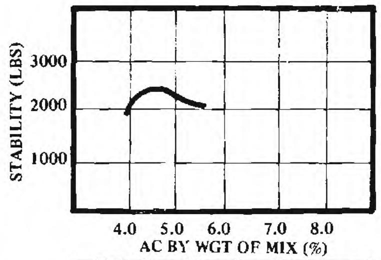
STABILITY (LBS.) : 2500

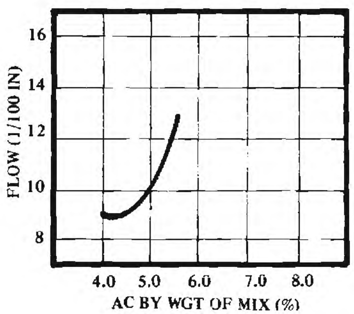
FLOW (1/100 IN): 9.7

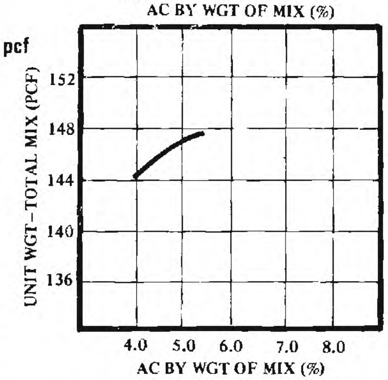
% VOIDS–TOTAL MIX: 9.2

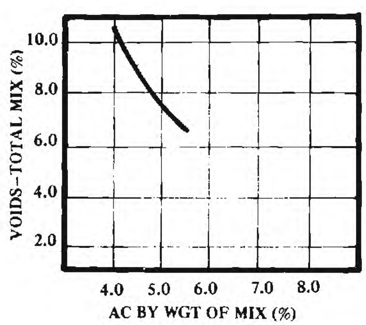
% VMA : 18.4

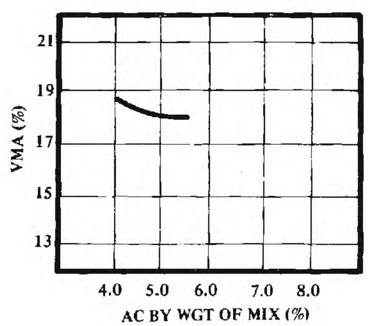
UNIT WGT–TOTAL MIX: 145.5 pcf











STONE/SOURCE: GRANITIC GNEISS / NORCROSS

50 BLOW MARSHALL MIX DESIGN

TYPE OF MIX: BLACK BASE

AGGREGATE GRADATION

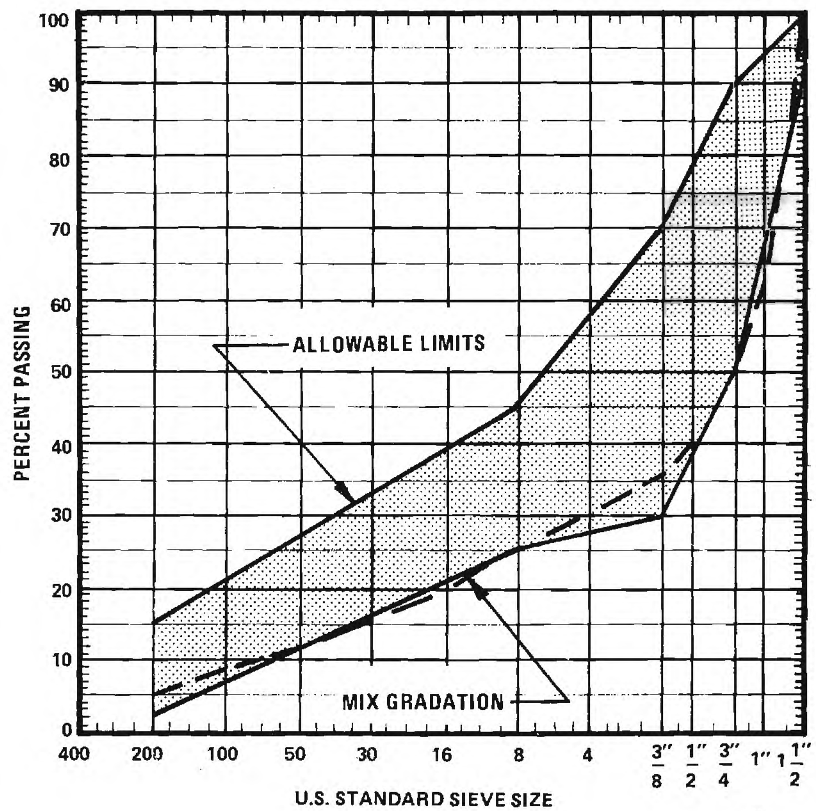
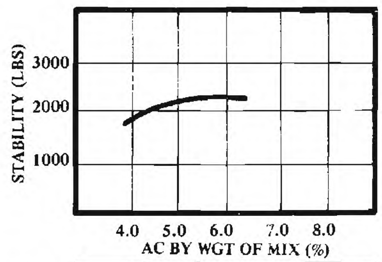
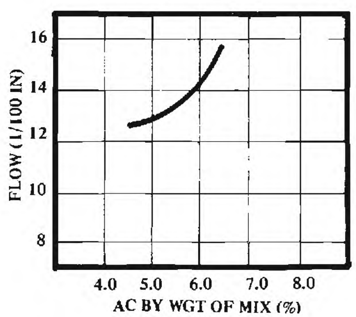


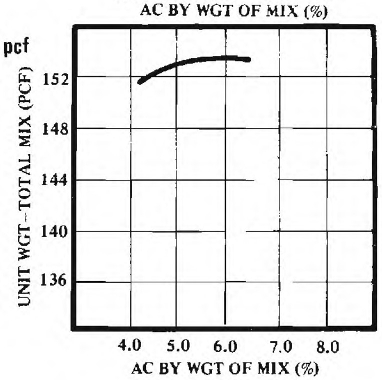
FIGURE 16. MARSHALL MIX DESIGN RESULTS FOR MIX 6–LIMESTONE BLACK BASE

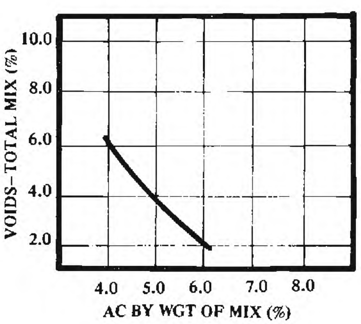
ASPHALT/SOURCE: AC–20 / SHELL OIL CO. (ATLANTA)
R&B SOFT. PT.: 124 °F
PEN. @ 77°F : 68
FILM THICKNESS: 9.5 MICRONS
MIXING TEMP : 285–295° F
COMPACTING TEMP : 262–278°F

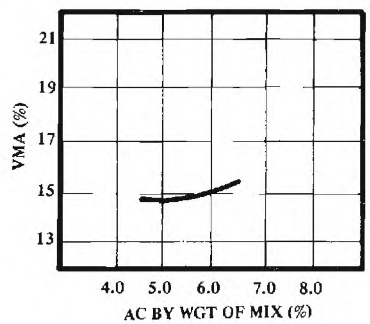
OPTIMUM % AC: 4.8
STABILITY (LBS.) : 2150
FLOW (1/100 IN): 12,7
% VOIDS–TOTAL MIX: 4.5
% VMA : 14.9
UNIT WGT–TOTAL MIX: 152.3 pcf











STONE/SOURCE: LIMESTONE / DALTON
50 BLOW MARSHALL MIX DESIGN
TYPE OF MIX: GEORGIA BLACK BASE

AGGREGATE GRADATION

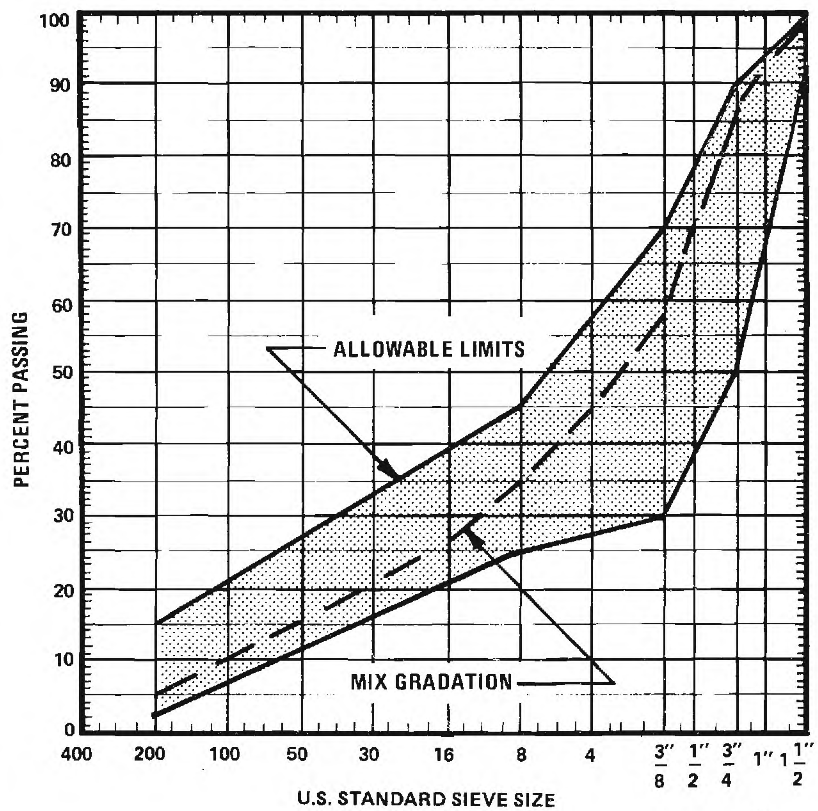
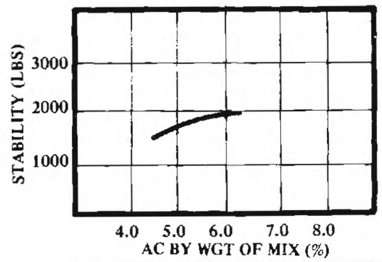
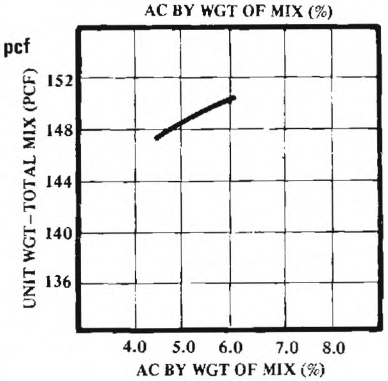


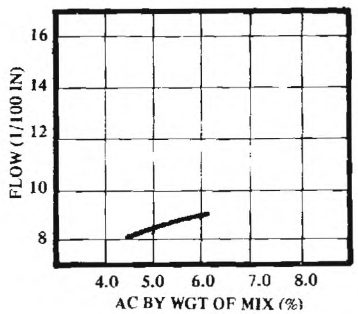
FIGURE 17. MARSHALL MIX DESIGN RESULTS FOR MIX 7–50 BLOW MODIFIED B MIX

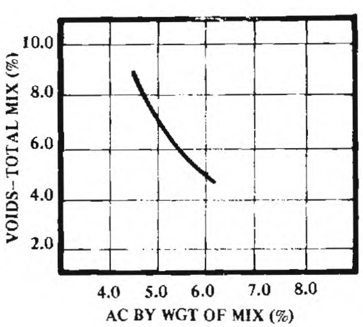
ASPHALT/SOURCE: AC–20 / SHELL OIL CO. (ATLANTA)
R&B SOFT. PT.: 124°F
PEN. @ 77°F : 68
FILM THICKNESS:
MIXING TEMP : 285–295°F
COMPACTING TEMP 262–278°F

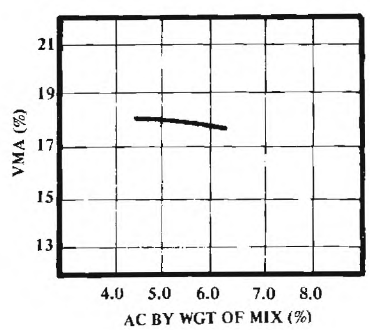
OPTIMUM % AC: 5
STABILITY (LBS.) : 1650
FLOW (1/100 IN): 8.3
% VOIDS–TOTAL MIX: 7.3
% VMA : 17.6
UNIT WGT–TOTAL MIX: 148.3 pcf











STONE/SOURCE: GRANITIC GNEISS / NORCROSS
50 BLOW MARSHALL MIX DESIGN
TYPE OF MIX: MODIFIED B

AGGREGATE GRADATION

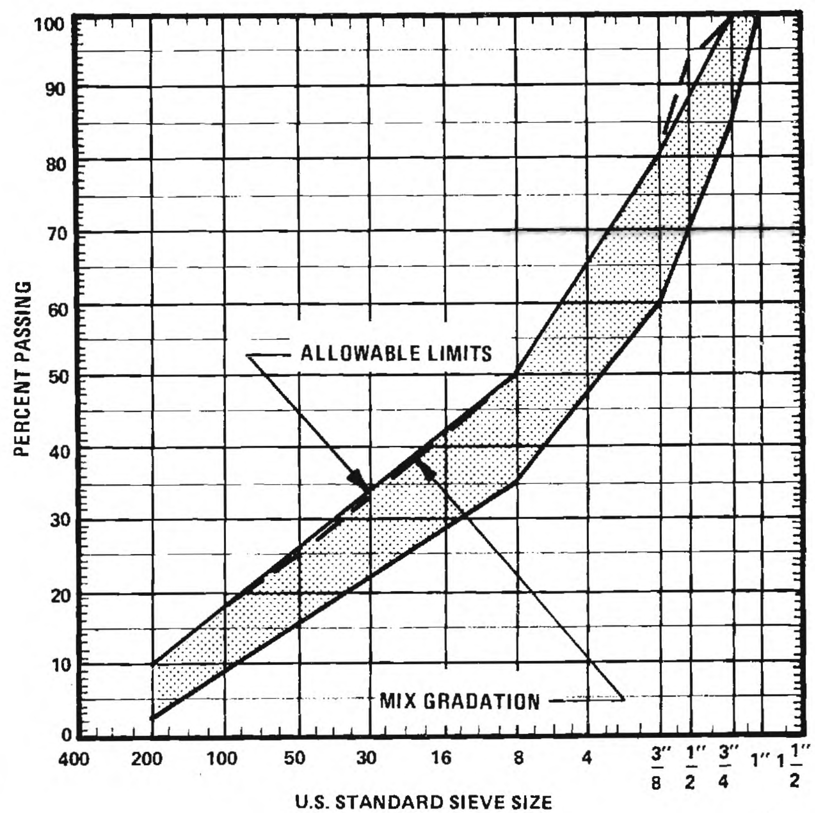
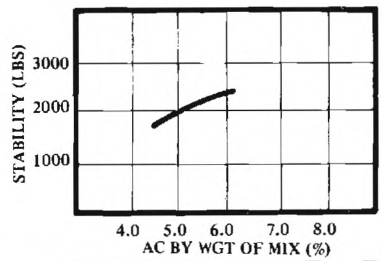


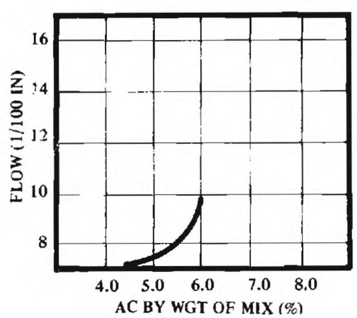
FIGURE 18. MARSHALL MIX DESIGN RESULTS FOR MIX8–75 BLOW MODIFIED B MIX

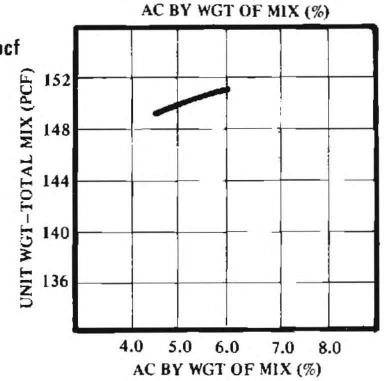
ASPHALT/SOURCE: AC–20 / SHELL OIL CO. (ATLANTA)
R&B SOFT. PT.: 124°F
PEN. @ 77°F :
FILM THICKNESS:
MIXING TEMP : 285–295°F
COMPACTING TEMP : 262–278°F

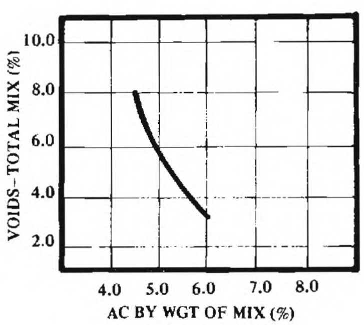
OPTIMUM % AC: 4.8
STABILITY (LBS.) : 1800
FLOW (1/100 IN): 7.8
% VOIDS–TOTAL MIX: 6.1
% VMA : 16.7
UNIT WGT–TOTAL MIX: 149.6 pcf

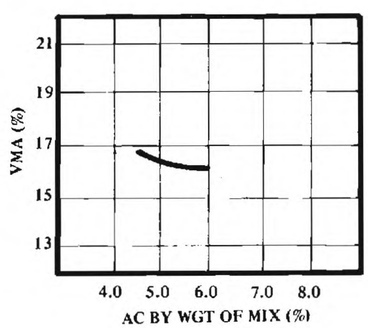
STONE/SOURCE: GRANITIC GNEISS / NORCROSS
75 BLOW MARSHALL MIX DESIGN
TYPE OF MIX: MODIFIED B



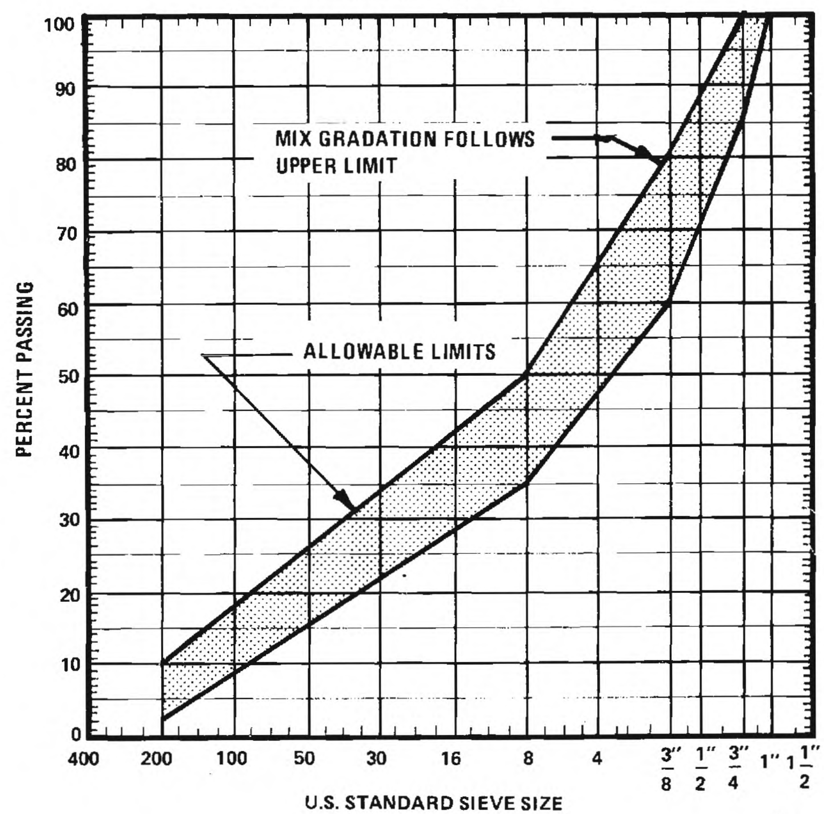








AGGREGATE GRADATION



detailed tabulation of information for the corresponding mix.

All aggregates used in this investigation were sieved, and the resulting stone sizes stored separately. Each specimen was prepared by weighing out the required material for each sieve size, and carefully blending them together.

The aggregate, asphalt, and mould were placed in an oven and heated to the temperature recommended by the Asphalt Institute [55] based on the viscosity of the asphalt cement. The mixing temperature for the specimens prepared using AC-20 asphalt cement was between 285°F and 295°F (140 and 146°C), and the compacting temperature was between 262°F and 278°F (128 and 137°C). For specimens prepared using AC-40 asphalt cement, the mixing temperature was between 305°F and 342°F (152 and 172°C), and the compacting temperature between 305°F and 315°F (152 and 157°C).

All specimens were moulded using a kneading type compactor. The kneading compactor is felt to produce laboratory specimens having a structure (orientation of aggregate) similar to that developed in the field during the rolling operation. The load foot of the compactor was held at the proper temperature by an internal heating coil. The anti-stripping agent, ADDELINE, was added to the mix at a rate of 0.5 percent of the liquid asphalt by weight. The heated mould and mould base were placed on the kneading compactor. The moulds, aggregate and asphalt were placed in an oven and heated for approximately 2-1/2 hours at the prescribed temperatures. The aggregate, asphalt and anti-stripping agent were then weighed in the proper proportions, and thoroughly mixed in a bowl so that all the aggregate particles were coated with asphalt cement. The asphalt concrete mixture was then immediately placed in the mould and compacted.

Cylindrical Specimens

The cylindrical specimens used in the repeated load triaxial tests and creep tests were 4 in. (102 mm) in diameter by 8 in. (203 mm) high. These specimens were compacted in a cylindrical steel mould using the kneading compactor. With the mould in place, the hot asphalt concrete mixture was spooned into the mould; as the mould was filled, the load foot was actuated downward so as to press down on the material in the mould one time between adding each spoonful of mixture. Filling the mould and compacting the specimen required approximately five minutes and took approximately 60 spoonfuls of material. This kneading action compacted the specimen to within 1/8 in. (3.2 mm) of the finished specimen height.

A circular piece of filter paper cut to fit the inside diameter of the mould was placed on top of the compacted specimen and a loading head was positioned on top of the filter paper. The entire mould assembly was immediately placed in a testing machine. A static load was placed on the specimen to level the top and to finish compacting it to the specified height of 8 in. (203 mm). The specimen was loaded on the top and bottom by two floating pistons to minimize end effects. After cooling under the static load, the specimen was extruded from the mould and then measured and weighed. A China marker was used to mark the sample number and circumferential lines 2 in. (51 mm) from the ends (102 mm apart) later used for positioning the LVDT clamps on the specimen.

Beam Specimens

The beam specimens used in this study for the fatigue tests were 3 in. by 3 in. (76 by 76 mm) in cross-section and 20 in. (508 mm) long. All beams were compacted and mixed at the same temperatures as those used in preparing the cylindrical specimens. After heating, the beam mould was placed in the kneading compactor on a sliding rack. Since the loading foot of the compactor does not move laterally, the beam mould was moved manually in the sliding rack during the compaction operation. The aggregate, asphalt cement and anti-stripping agent were mixed together in preweighed amounts so that all aggregate particles were completely coated by the asphalt cement. The hot asphalt concrete mixture was then placed in the mould in four layers. Each layer was compacted by 3 to 4 passes of the compactor along the length of the beam. After all asphalt concrete was placed in the mould, a loading plate was positioned on top of the beam and loaded until a height of 3.0 in. (76 mm) was reached. This procedure also served to level the surface of the specimen.

The beam and mould were allowed to cool, and the mould was removed. After cooling, each specimen was measured, weighed and then stored on a surface ground steel plate. The specimens were stored on the flat surface so they would lie flat on the rubber pad used in the fatigue test. The use of the machined steel plates for storage of the beams was necessary to avoid inducing tensile strains in the beam before testing, and to give uniform subgrade support to the beam during the fatigue test. The specimen number, asphalt content, date of compaction and the future location of the loading foot were marked on each specimen.

A reference point for measuring the deflection of the centerline of the beam was then established by epoxy gluing a small aluminum tab on one side along the neutral axis at the midpoint of the beam. An SR-4 wire resistance strain gauge (BLH A9-4) was glued below the reference tab 0.1 in. (2.5 mm) above the bottom of the beam. The strain gauge was oriented parallel to the neutral axis to measure the maximum bending tensile strain in the beam. A strain gauge having a relatively long gauge length of 2.0 in. (51 mm) was used to minimize the effects of the relatively large aggregate present in the base course mixes.

A fast setting epoxy glue was used to bond both the aluminum tabs and strain gauges to the beams. The glue dried in approximately 5 minutes and later permitted relatively easy removal of the reference tab and strain gauge from the beam.

CHAPTER 5

EQUIPMENT AND TEST PROCEDURES

Introduction

Repeated load testing was used in this study to evaluate the rutting and fatigue characteristics of nine asphalt concrete base course mixes. The results of the repeated load triaxial test were compared with creep test results to evaluate the rutting characteristics of the asphalt concrete. In the repeated load triaxial test the deviator stress, $\sigma_1 - \sigma_3$, was cycled while the confining stresses, σ_3 was held constant.

The fatigue test was performed by placing the beams on a rubber subgrade and applying a repeated load at the center. The fatigue test results were used to determine the relative fatigue life of the mixes studied, and to estimate the fatigue life of a typical structural pavement section. Specimens were tested between 7 and 14 days after preparation. In the fatigue test a cyclic load was applied at the center of the beam over a width of 1.25 in. (32 mm). The beam specimens were not subjected to stress reversals during testing. A pneumatic loading system was used to apply the cyclic loading in both the rutting and fatigue tests. The cyclic load was applied by means of a Bellofram cylinder into which air was cycled by a 5-way spool valve. The movement of the spool valve was controlled by two solenoid pilot valves that were actuated by the electrical signals from an electronic cyclic timer. Both the duration of load pulse and the rest time between pulses could easily be set using the electronic cyclic timer. The load pulse used in the repeated load triaxial and fatigue tests had a duration of 0.06 seconds and approximately a haversine shape. The load pulse was applied 45 times per minute.

A multi-layered elastic analysis of typical pavement structures was used to determine appropriate axial and confining stresses to use in the rutting test. Specimens were subjected to only compressive stress states in the triaxial test. The rutting test specimens were subjected to 100,000 repetitions in generally the confined state. The creep tests to evaluate rutting were run unconfined at a constant axial stress (usually 15 psi) for 10,000 seconds. The fatigue tests were run until the beam failed at a constant load level generally varying from 100 to 250 lbs. (440 to 1110 N).

All electronic instrumentation was carefully calibrated to ensure accurate test results. The linear variable differential transducers (LVDT's) used to measure deflections were all calibrated with a micrometer calibration device accurate to 0.0001 in.

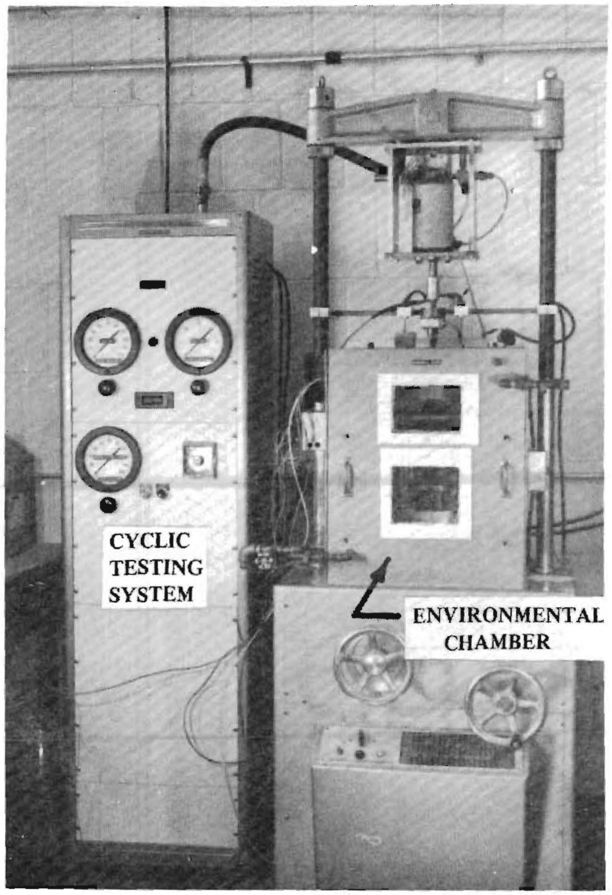
(0.0025 mm). The LVDT calibrations were checked before each test by using a steel measuring block having a thickness of 0.030 in. (0.76 mm). The load cells were statically calibrated by applying a load of known weight and recording the output from the load cell. The SR-4 strain gauges were statically calibrated by axially deforming a cylindrical asphalt concrete specimen having a strain gauge glued along its longitudinal axis. The specimen deflection was measured with a dial indicator and the average specimen strain was calculated and then correlated with the strain measured using the SR-4 strain gauge.

Repeated Load Triaxial Test

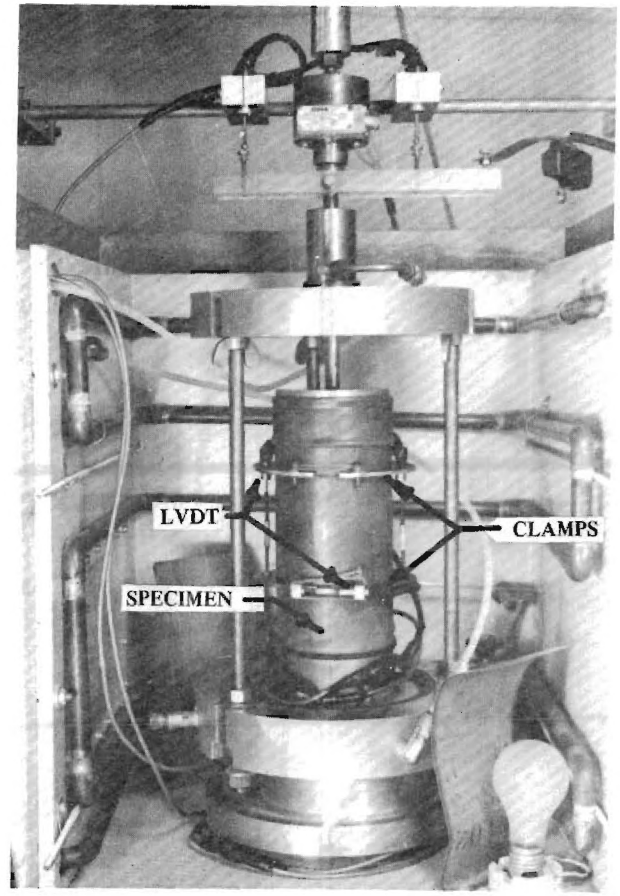
The repeated load triaxial test was used to compare rutting characteristics of the different asphalt concrete mixes. Tests were performed on both unconfined specimens and specimens subjected to confining pressures up to 30 psi (207 kN/m^2). Deviator stresses varied from 8 to 40 psi (55 to 276 kN/m^2) depending upon the confining pressure used. Temperature was varied from 85^oF to 105^oF (29 to 41^oC), with the standard test temperature being 95^oF (35^oC). Ngowtrakul [47] has found that approximately half of the rutting in Georgia occurs at pavement temperatures above 95^oF (35^oC) and half occurs below this temperature. Therefore, most repeated load triaxial tests were performed at a temperature of 95^oF (35^oC) which is approximately the mean pavement temperature for the rutting which occurs in Georgia.

Specimens 4 in. (102 mm) in diameter were tested in a 6 in. (152 mm) diameter triaxial cell enclosed in a controlled environmental chamber (Fig. 19). Axial strain was measured by placing two clamps on the specimens as illustrated in Fig. 20 and measuring the movement between these clamps using two small AC LVDT's. The LVDT's used to measure the axial deformation were wired together to give the average specimen movement between clamps. This instrumentation arrangement minimizes the effects of possible tilting of the specimen. Radial strain was measured by a single LVDT oriented horizontally in the plane of the diameter of the specimen at the open end of the lower clamp (Fig. 20). This LVDT measured a deformation directly related to the change in diameter of the specimen. The two clamps were placed at a level one quarter of the distance in from each end of the specimen to minimize end effects. To minimize the size and weight of the measurement devices attached to the clamps, 0.375 in. (9.5 mm) diameter AC type LVDT's were used. The outputs from the inside axial and radial transducer measuring systems were recorded on a Hewlett-Packard two channel, strip chart recorder.

The total axial specimen deformation was measured by a pair of DC LVDT's which



(a) GENERAL TESTING SYSTEM



(b) SPECIMEN AND LVDT CLAMPS

FIGURE 19. REPEATED LOAD TEST APPARATUS

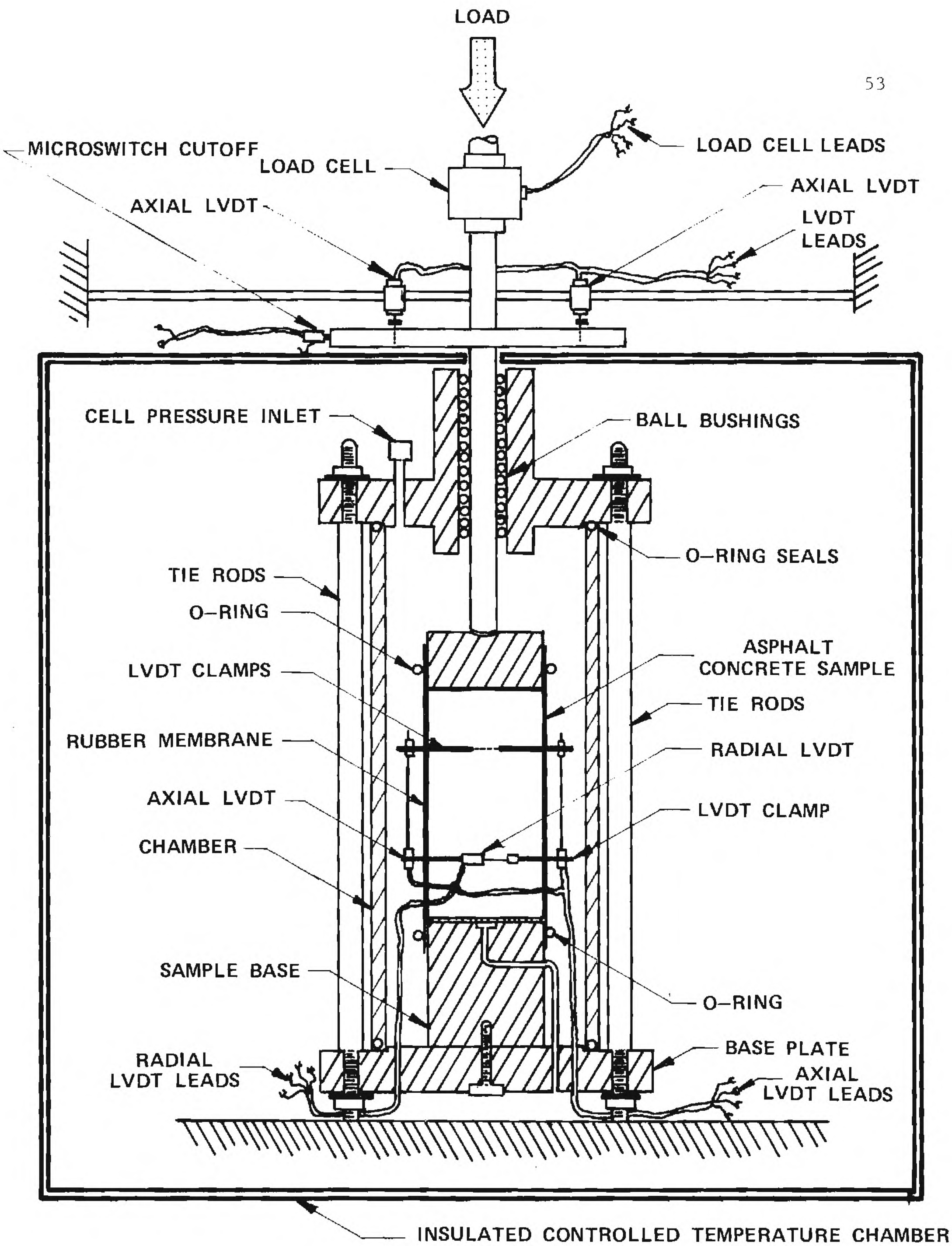


FIGURE 20. CONTROLLED TEMPERATURE REPEATED LOAD TRIAXIAL SYSTEM FOR DYNAMIC TESTING OF ASPHALT CONCRETE

reacted against a Lucite clamp attached to the loading piston outside the environmental chamber (Fig. 20). The output from these transducers was recorded on a Hewlett-Packard X-Y recorder. Load was measured by a 2,500 lb. (11 kN) capacity load cell and recorded on a two channel strip chart recorder.

The test procedure used for the repeated load triaxial test is summarized as follows: Each specimen was first carefully examined to assure that it was free from defects such as excessive voids due to the presence of large aggregates, and that both ends were flat and parallel. A rubber membrane was then placed around the sides of the specimen. The specimen was positioned on top of a bronze porous stone resting on the bottom loading platen of the triaxial cell. Top end friction on the specimen was minimized by rubbing a silicone lubricant over the top of the specimen. A thin Teflon pad was placed between the end of the specimen and the top platen. The rubber membrane was then pulled up over the top platen, and rubber O-rings were used to seal the membrane to the top and bottom platens.

The inside LVDT clamps were placed around the rubber membrane, and the LVDT probes were set at approximately the null voltage output position. The clamps were placed on the specimen tightly enough to prevent slippage during the test, but not so tight as to exert excessive additional confining stress on the specimen. Once the LVDT's were in place and adjusted, the triaxial chamber was assembled. The environmental chamber was then placed around the cell, and the loading piston inspected to see that it was in alignment with the top platen on the specimen. The top cross-arm of the loading system was lowered so that a small seating load was applied to the specimen. The triaxial chamber and enclosed specimen were maintained at the desired testing temperature overnight in order for the specimen to reach the desired temperature.

The pens on the recorder monitoring the LVDT deflections were centered and the test started. Specimen deformation was measured continuously for the first 10 repetitions, and then for a short time at approximately 100, 1000, 10000, 50000 and 100000 load repetitions. After approximately 100,000 repetitions, the test was terminated. During the test the chamber temperature, cell pressure, pilot valve pressure, deviator stress and load pulse time were observed periodically to insure proper adjustment.

The resilient modulus, plastic (permanent) strain and Poisson's ratio as a function of the number of load repetitions were obtained from this test.

Creep Test

The equipment for running the creep test consisted of a triaxial cell, a pneumatic constant-stress loading system and an environmental chamber as illustrated in Fig. 21. The axial load was measured with a 2,500 lb. (11 kN) load cell and recorded on a Sanborn, two channel strip chart recorder. The axial deformation was measured using a single d.c. LVDT and recorded on a Mosely X-Y recorder. To minimize friction, both ends of the specimen were lightly coated with a silicone lubricant to fill the voids, and the lubricant was sprinkled with powdered graphite. A thin Teflon pad was then placed on each end of the sample, and the sample was put into the triaxial chamber without the Lucite confinement cylinder. As recommended previously [40], a 15 psi (2.1 kN/m^2) axial stress was applied to the sample which is within the linear range of usual asphalt concrete mixes. The specimens were stored overnight in the environmental temperature chamber at 95^oF (35^oC) and tested at that temperature.

The specimen was subjected to a single step loading using the pneumatic testing system. Specimen deflection was recorded at 1, 4, 10, 40, 100, 400, 1000, 4000 and 10,000 seconds to obtain the permanent strain in the specimen as a function of time. Load and temperature were monitored throughout the test to insure proper testing conditions.

Upon completion of the creep test, the rut depth was calculated for an 18 kip (80 kN) single axle load after an equivalent of 10^6 repetitions at an average pavement temperature of 95^oF (35^oC) using the Shell Method as described by Van de Loo [40]. To calculate rut depth using this approach, the following properties of the asphalt cement must be known: viscosity, penetration, ring and ball softening point and the relationship between strain in the specimen and time. Knowing these physical properties, the rut depth can be calculated after the stiffness of the mix has been determined experimentally.

Fatigue Test

A beam fatigue test was used to evaluate the fatigue characteristics of the asphalt concrete base mixes. In this study, an asphalt concrete beam placed on a rubber subgrade was used to simulate field support conditions. This test also eliminates the problem of beam weight which can affect the results of unsupported beam fatigue tests.

The fatigue test equipment (Fig. 22) consisted of load frame, a 4 in. (102 mm) thick rubber pad supporting the beam, and a pneumatic loading system. The fatigue specimen and rubber support were enclosed within an environmental chamber maintained at $80^oF \pm 1^oF$ (27^oC). The rubber pad had a Durometer reading of 40 and a modulus of subgrade reaction

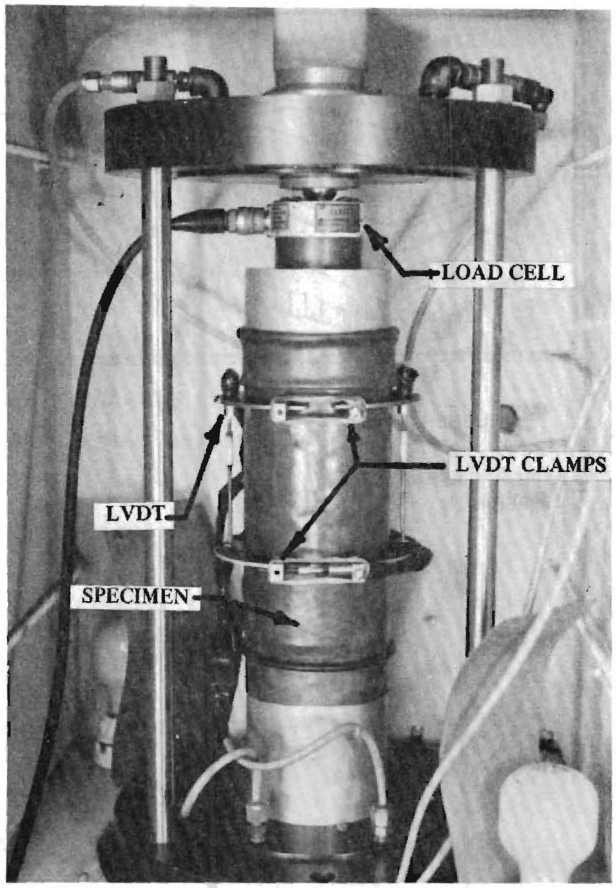
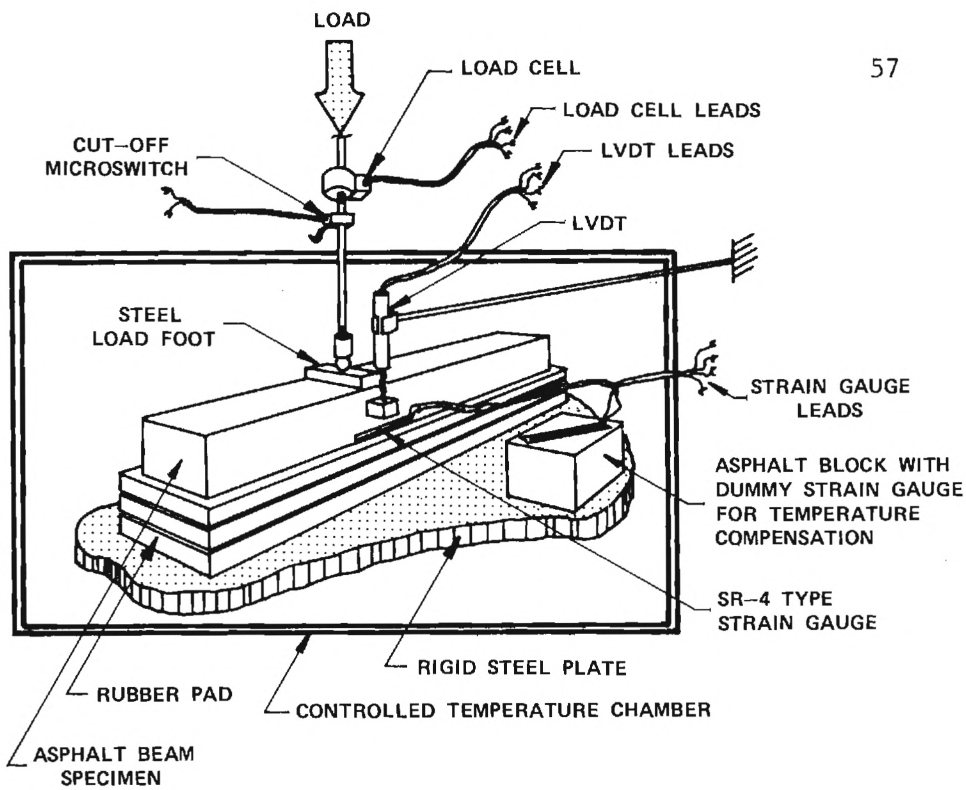
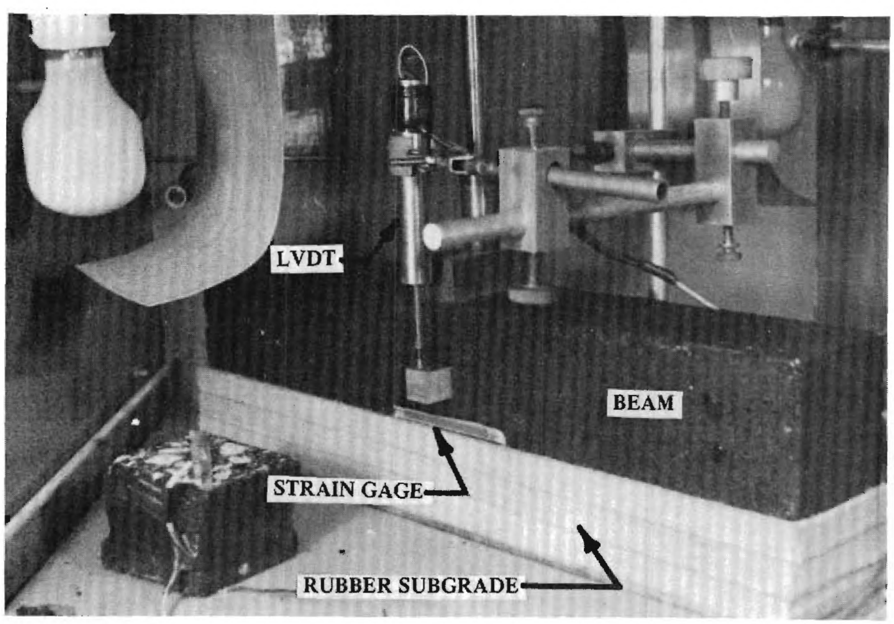


FIGURE 21. CREEP TEST APPARATUS



(a) SCHEMATIC



(b) CLOSE–UP PHOTOGRAPH

FIGURE 22. FATIGUE TEST APPARATUS

of 284 pci (7,861 gm/cc). The rubber pad was supported on the surface ground base of the load frame.

The load was applied to the center of the beam using a rigid steel plate which was 1.25 in. (32 mm) wide, 3 in. (76 mm) long and 1 in. (25 mm) thick. A SR-4 strain gauge was epoxy glued to the side of each beam in a direction parallel to its axis 0.10 in. (2.5 mm) above the bottom. The strain gauge was located symmetrically below the center of the load. Another SR-4 strain gauge was glued to an asphalt concrete block inside the environmental chamber. This temperature compensating strain gauge was used to eliminate any errors in measuring the strain in the beam that could be caused by possible small changes in temperature occurring in the chamber and differences in temperature from the calibration value.

The deflection of the center of the beam was measured with a single d.c. LVDT and recorded on a Sanborn, two channel strip chart recorder. The constant load applied to the top of the beam was measured using a 2,500 lb. (11 kN) capacity load cell and recorded on a Sanborn, two channel strip chart recorder. To determine the number of load repetitions to failure, an automatic timing system was developed to measure at one to 3 hour time intervals the centerline deflection of the beam. This system consisted of a mechanical cyclic timer wired into the paper drive motor on the Sanborn recorder and also to the 110 volt AC power cord leading to the 24 volt DC power supply for the LVDT.

Before each fatigue test was begun, the rubber pad was removed and the supporting surface beneath was thoroughly cleaned to insure continuous contact at the interface. After cleaning, the pad was replaced and an asphalt concrete beam specimen was carefully centered on the rubber pad. The strain gauge leads were then soldered to the connections on the strain gauge, and the system checked for continuity. The load foot was placed inside the previously placed reference marks on top of the beam. The LVDT with the probe in place was positioned against the aluminum tab glued to the side of the beam and was vertically aligned.

For a given series of tests, the desired number of load repetitions to failure for each test was selected to define the relationship between load level (and hence strain level) and the number of repetitions to failure. The corresponding magnitude of load to cause failure at this number of repetitions was estimated from the results of previous fatigue tests. The deflection under the load at the center of the beam and the radial tensile strain were measured at 100, 200, 500, 1,000 and approximately 1,500 repetitions. The tensile strain measured at 1,000 load repetitions was used (with the exception of a few tests) in interpreting the test results. The strain at 1,000 load repetitions was used to allow stabilization of the response of the beam to occur. After 1,500 repetitions, the strain gauge and epoxy glue were removed from the side of the beam to eliminate any strengthening effect. After this the automatic deflection measuring system was used to obtain the centerline beam deflection at periodic time intervals. A cutoff switch located on the loading piston was installed to automatically stop the test upon failure of the beam. Cracking of the beam was generally initiated in the bottom directly beneath the load. This crack rapidly propagated upward, and upon failure the beam would usually separate into two parts. Failure of the beam was defined by the relatively sudden large increase in centerline deflection of the beam as indicated by the automatically recorded beam deflections. Actual observations of the failure of several beams indicated the approach used was sufficiently accurate for establishing the number of repetitions to failure. The information from the strip chart recorders, along with calibration constants for the various electronic measuring devices, were used to calculate the radial tensile strain in the bottom of the beam, the applied load and the tensile bending modulus of the asphalt concrete.

CHAPTER 6

TEST RESULTS

Introduction

Fatigue and rutting tests (both creep and repeated load triaxial tests) were performed on both black base and Modified B base course mixes. For most mixes, fatigue and rutting tests were performed at three asphalt contents in the vicinity of 4.2, 4.8 and 5.5 percent. The variation in asphalt contents investigated was selected to bracket the range of values likely to be used by the Georgia Department of Transportation for usual base course construction. Asphalt concrete specimens were prepared using both 50 blow and 75 blow Marshall Mix Designs. The 50 blow Marshall mix is presently specified by the Georgia Department of Transportation. Specimens at each selected asphalt content were prepared to have the same density and voids content as defined by the Marshall Mix Design curve.

A summary of the materials tested is given in Table 7, p. 37. Except as indicated otherwise, a significant portion of the tests on the black base were performed on a "standard" 50 or 75 blow Marshall mix. This mix was prepared using AC-20 asphalt cement and a crushed granitic gneiss aggregate. The characteristics of these two mixes are completely described in Figs. 11 and 12, pps.39 and 40. The granitic gneiss aggregate used was well-graded and had a maximum size of 1-1/2 in. (38 mm) with 42 percent passing the No. 4 sieve and 4 percent passing the No. 200 sieve. The optimum asphalt content reported for the 50 blow Marshall Mix Design was 4.8 percent (4.5 percent voids) and the optimum for the 75 blow Marshall mix was 4.5 percent (3.8 percent voids).

Several additional black base mixes were investigated. In addition to using AC-20 asphalt cement, a 50 blow mix was also prepared using an AC-40 asphalt cement. Also the effect was investigated of using a crushed limestone (Fig. 16) having the same gradation as the "standard" mix. Mixes having a coarse and fine gradation were studied to determine the influence of gradation on the fatigue and rutting characteristics. These black base mixes were prepared using a granitic gneiss aggregate. The finer mix (Fig. 14) had a 1-1/2 in. (38 mm) maximum aggregate size, 97 percent passing the 1 in. (25 mm) size, 55 percent passing the No. 4 sieve and 14 percent passing the No. 200 sieve. The mix with the coarse gradation (Fig. 15) had a maximum size of 1-1/2 in. (38 mm), 63 percent passing the 1 in. (25 mm) sieve, 30 percent passing the No. 4 sieve, and 5 percent passing the No. 200 sieve. Both these mixes were prepared using the AC-20 asphalt cement and the granitic gneiss aggregate. The effect on fatigue and rutting properties was also studied of portland cement and flyash mineral filler.

The 50 and 75 blow Modified B Mixes (Figs. 17 and 18) investigated had a gradation which followed the finer allowable limits of the specification. This material had 100 percent passing the 3/4 in. (19 mm) sieve, 66 passing the No. 4 sieve, and 10 percent passing the No. 200 sieve. The same crushed granitic gneiss aggregate was used as in the other tests. The specimens were prepared using an AC-20 asphalt cement.

Fatigue Test Results

The fatigue test results are presented in Appendix C in terms of repetitions to failure as a function of both the constant load applied to the beam (Figs. C-1 to C-11) and the maximum tensile bending strain (Figs. C-12 to C-22).

The bending stiffness of each mix as calculated from the measured tensile strain in the beam fatigue test is shown in Figs. C-23 to C-33. A comparison of these figures show, depending on the characteristics of the mix, that the bending stiffness (bending modulus of elasticity) of the asphalt concrete mixes varies from approximately 40,000 psi to 150,000 psi (2.76×10^5 to 1.03×10^6 kN/m^2) at one million load repetitions and a temperature of 80^oF (27^oC). The method of calculating the bending modulus from the results of the beam fatigue test is summarized in Appendix B. As indicated by equation (B-1), tensile strain is a function of both the applied load and the bending stiffness of the asphalt concrete beam. Under a given loading such as would occur in the pavement, the mix having the higher stiffness would have a lower strain than the mix with a lower stiffness. Therefore, the fatigue curves based on tensile bending strain cannot be directly compared with each other unless the bending stiffness at the desired number of load repetitions is approximately the same.

A direct comparison of fatigue test results can be made using the measured relationships between load and repetitions to failure (Figs. C-1 to C-11). An alternate method that can also be used is to calculate using layered theory the tensile strain that would occur in a pavement using the measured bending stiffness of the mix. The number of repetitions required to cause failure is then determined directly from the laboratory fatigue curve using the the calculated tensile strain. Comparisons of the number of repetitions required to cause failure for a typical pavement are given in Chapter 7 using both the load and layered system methods of interpreting the test results.

Rut Test Results

Rutting was evaluated using both the repeated load triaxial test and also the Shell creep test. Both repeated load and creep tests were performed at $95^{o}F$ ($35^{o}C$). The repeated load triaxial test results are given in Figs. 23 to 30 in terms of permanent (plastic) strain as a function of the asphalt content. Most of these tests were performed at a confining pressure of 5 psi (34 kN/m^2) and a deviator stress of 25 psi (172 kN/m^2). This stress condition is reasonably close to the one which would be expected to occur near the center of an asphalt concrete layer. For this stress state the rut depth in a 10.5 in. (267 mm) asphalt concrete layer would be approximately 4.5 times the measured permanent strain.

The creep test results are presented in Figs. 31 to 33 in terms of predicted rut depth as a function of asphalt content. The rut depth is predicted using the Shell Method for 1 million, 18 kip (80 kN) axle loads applied at a pavement temperature of $95^{o}F$ ($35^{o}C$). The correction factor C_m which is described in Appendix A was considered to be 1.8 in the computations for rut depth using the creep test results.

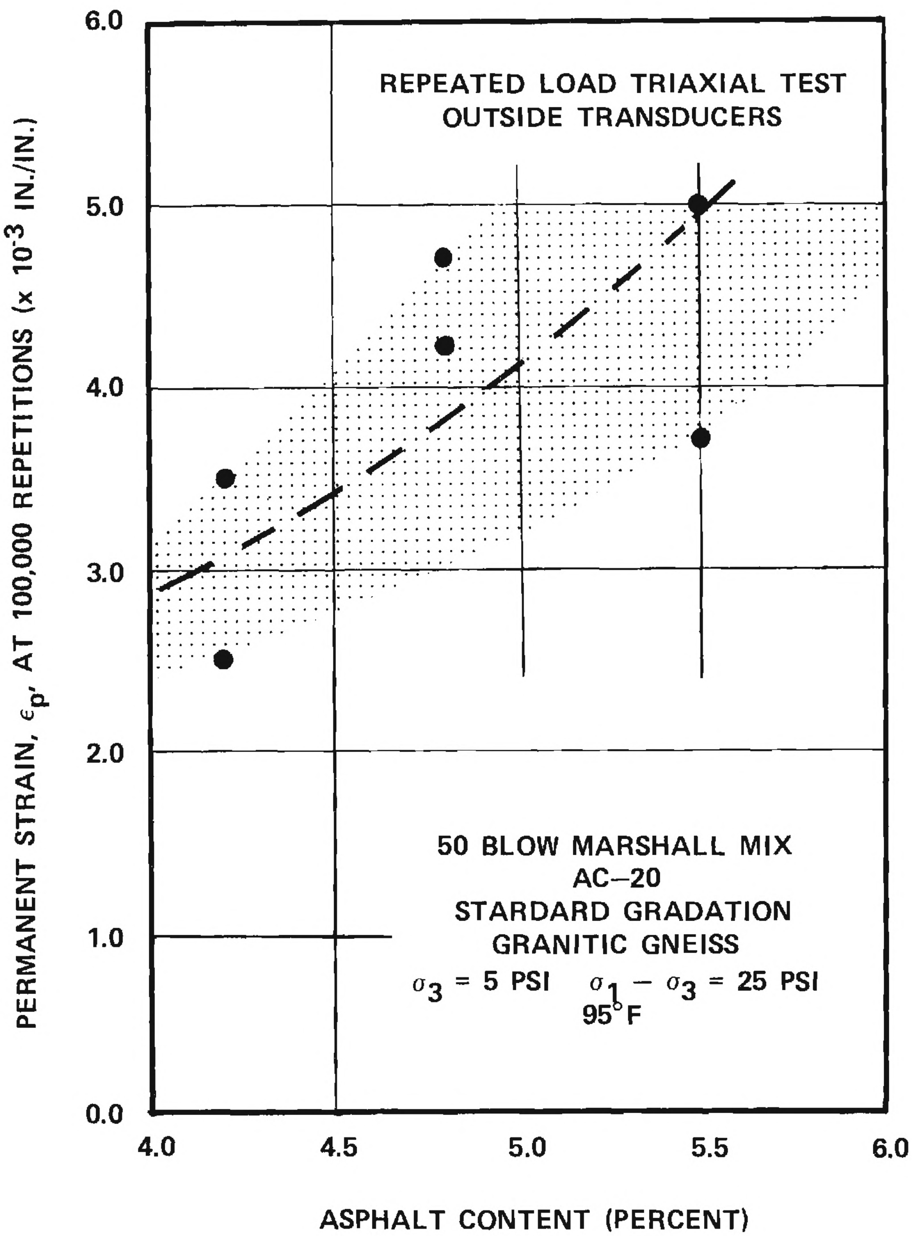


FIGURE 23. INFLUENCE OF ASPHALT CONTENT ON PERMANENT STRAIN IN 50 BLOW BLACK BASE FROM REPEATED LOAD TEST (σ_3 = 5 PSI, $\sigma_1 - \sigma_3$ = 25 PSI)

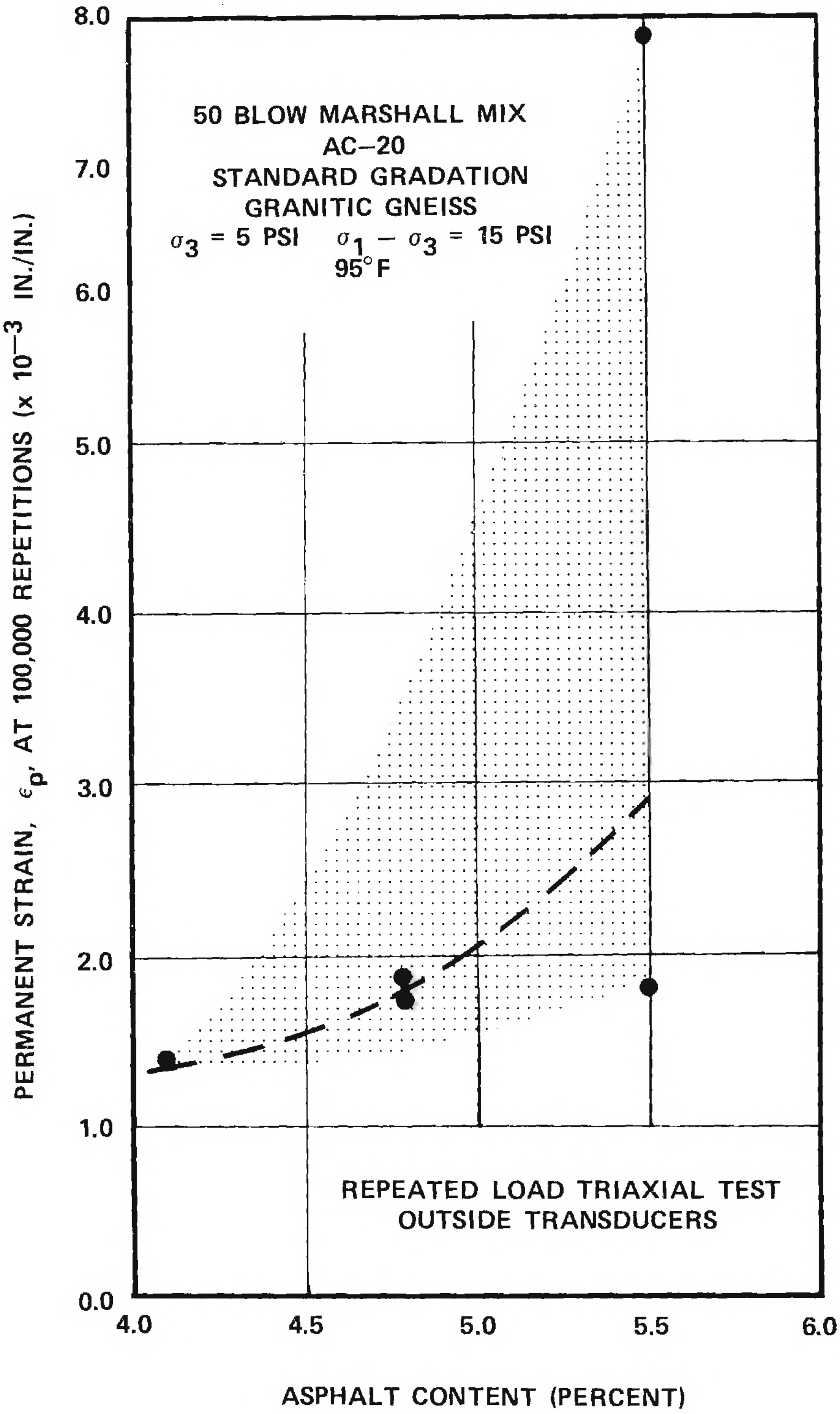


FIGURE 24. INFLUENCE OF ASPHALT CONTENT ON PERMANENT STRAIN IN 50 BLOW BLACK BASE FROM REPEATED ROAD TEST (σ_3 = 5 PSI, $\sigma_1 - \sigma_3$ = 15 PSI)

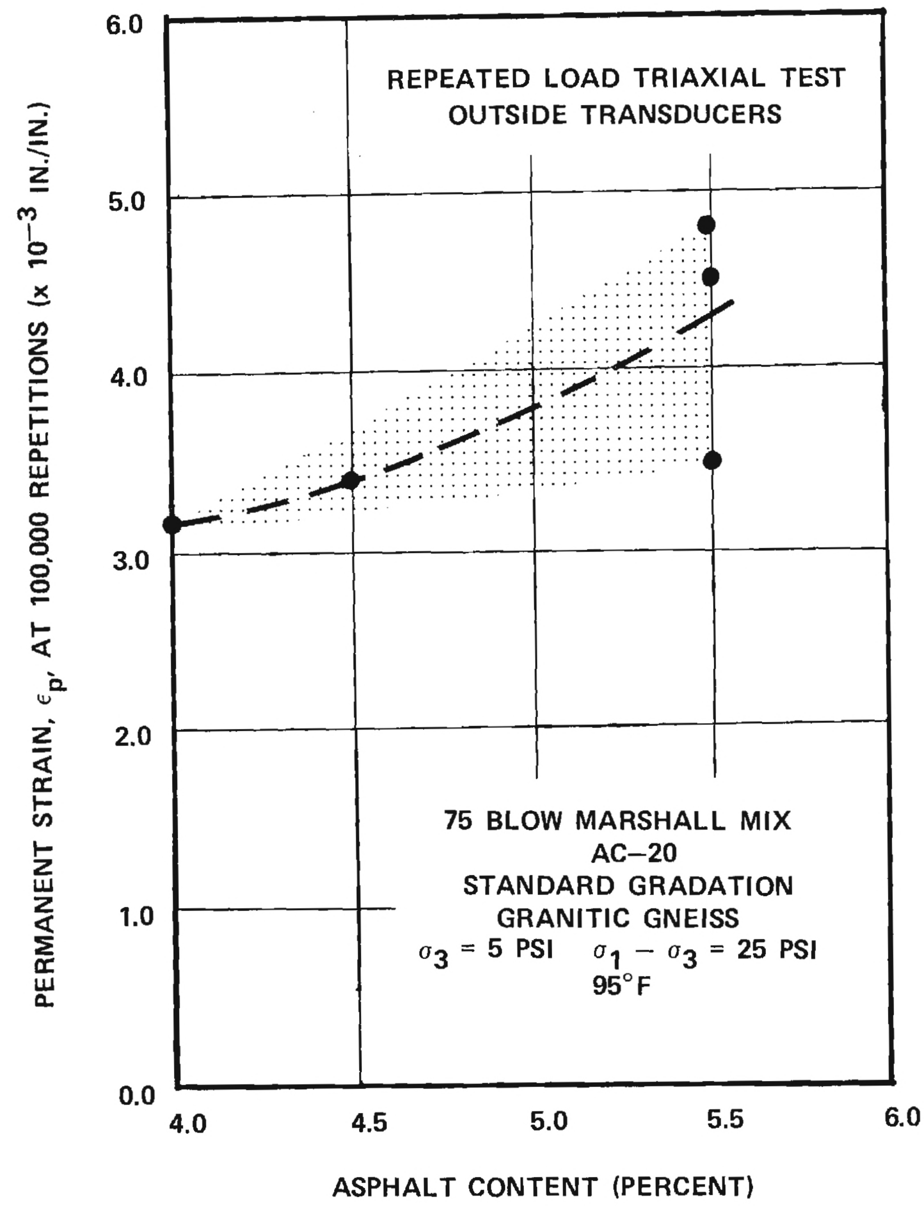


FIGURE 25. INFLUENCE OF ASPHALT CONTENT ON STRAIN IN 75 BLOW BLACK BASE MIX

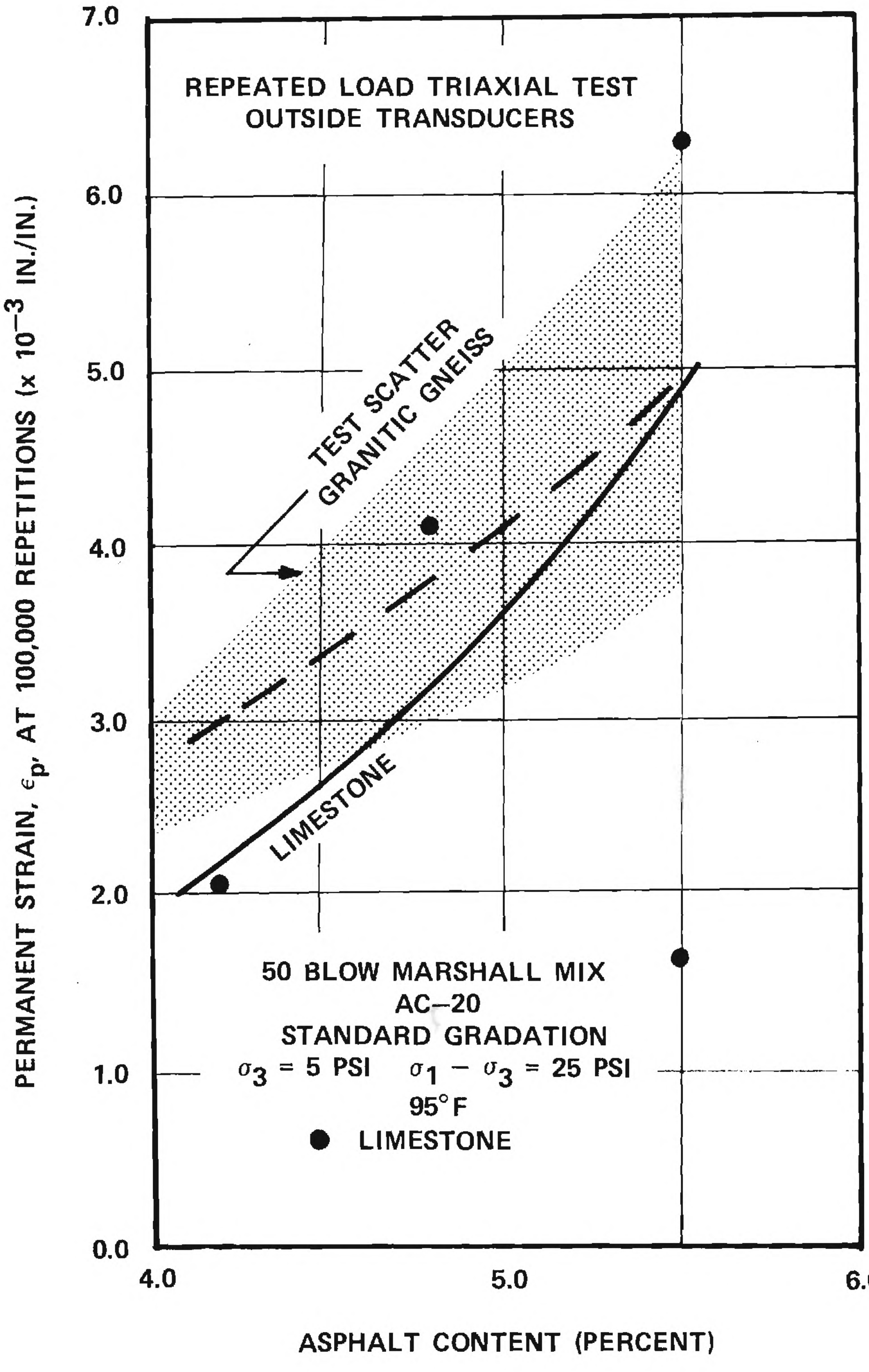


FIGURE 26. INFLUENCE OF AGGREGATE TYPE ON PERMANENT STRAIN IN BLACK BASE MIX

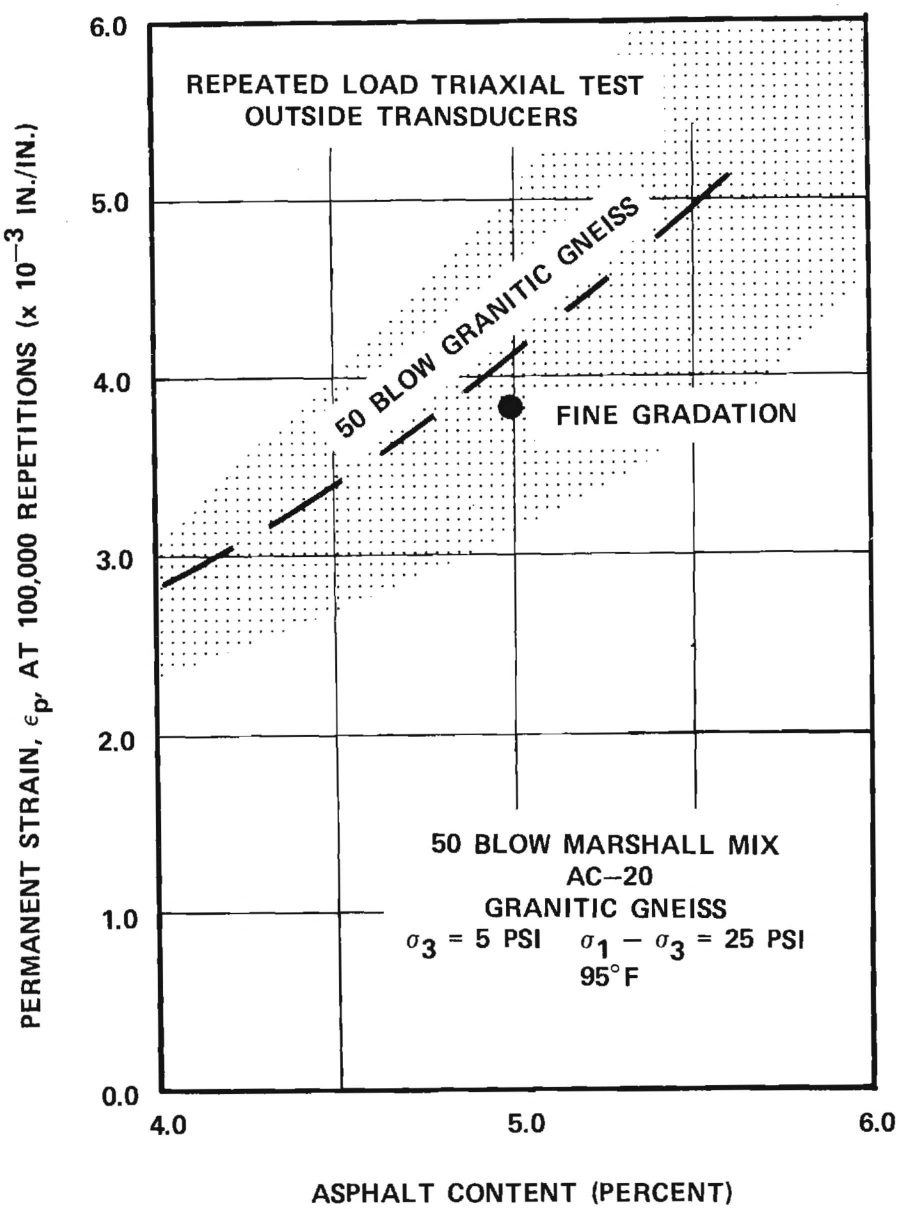


FIGURE 27. INFLUENCE OF GRADATION ON PERMANENT STRAIN IN 50 BLOW BLACK BASE MIX

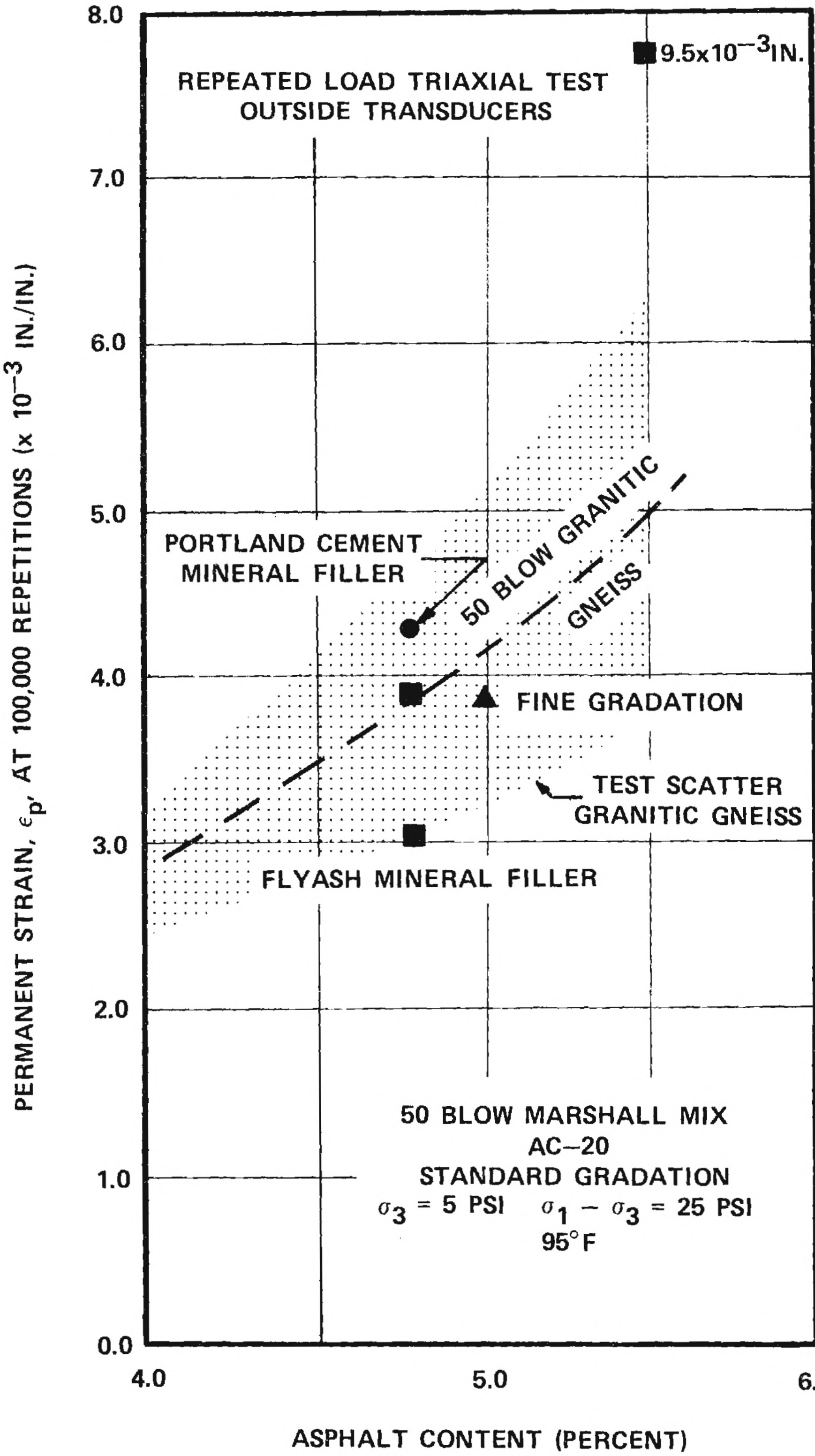


FIGURE 28. INFLUENCE OF MINERAL FILLER ON PERMANENT DEFORMATION IN A 50 BLOW BLACK BASE MIX

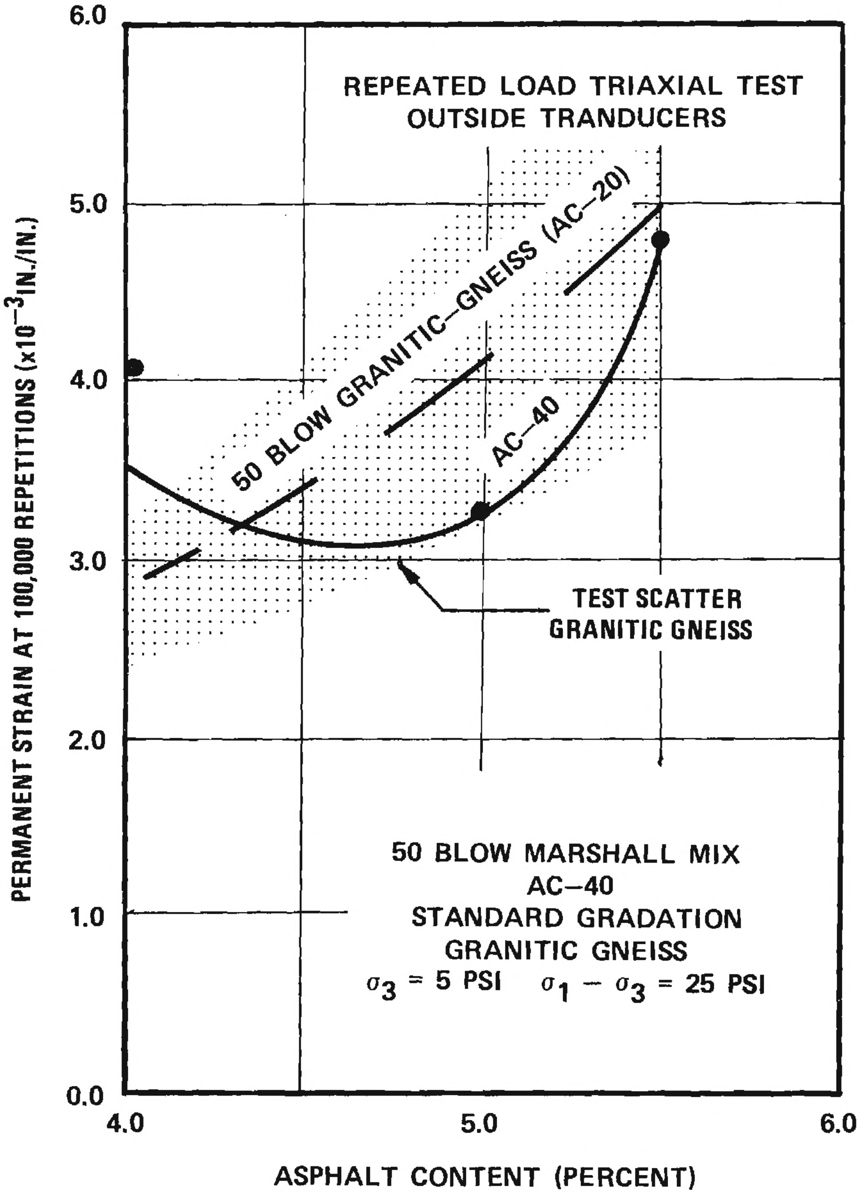


FIGURE 29. EFFECT ON PERMANENT STRAIN OF USING AN AC-40 ASPHALT CEMENT IN 50 BLOW BLACK BASE MIX

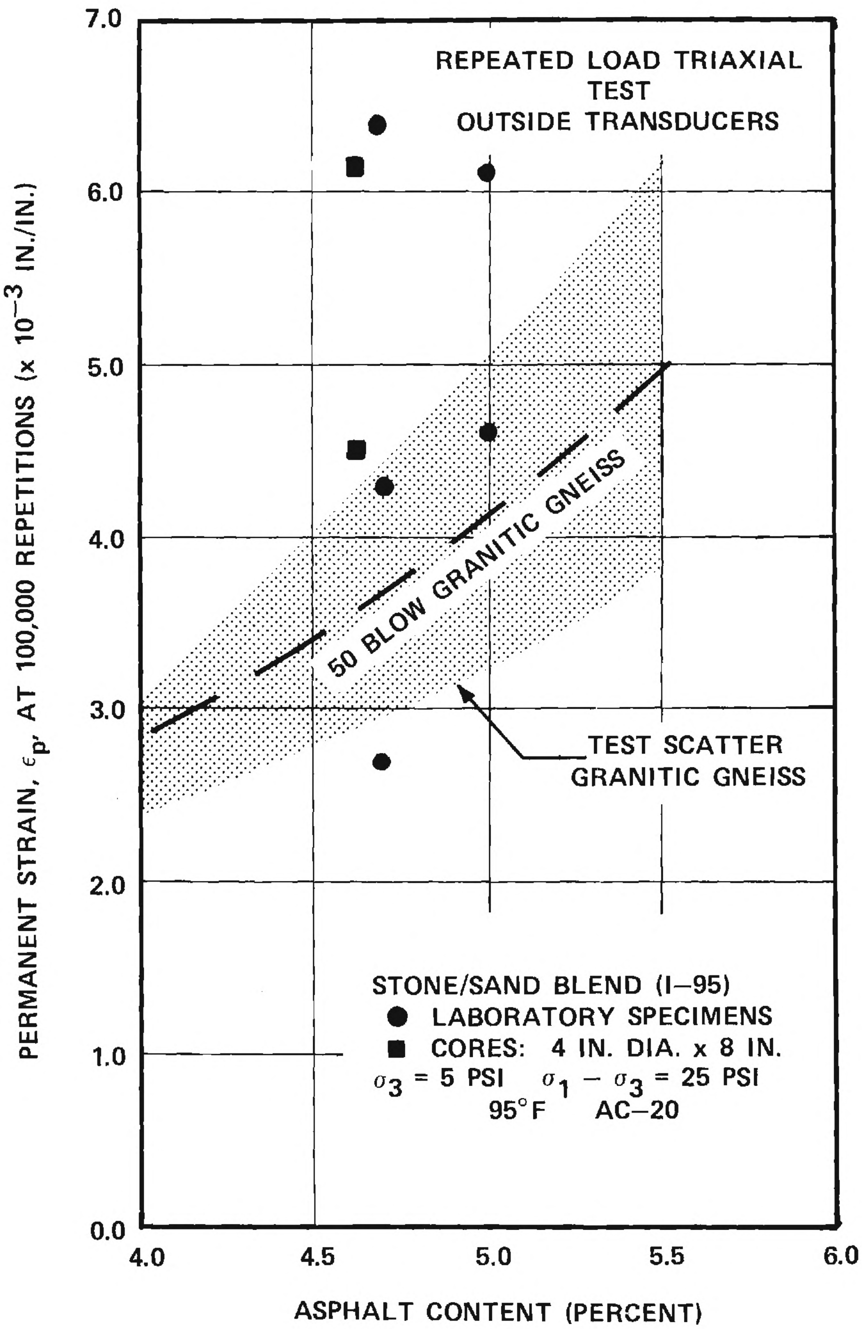


FIGURE 30. INFLUENCE OF ASPHALT CONTENT ON PERMANENT STRAIN IN STONE–SAND BLEND (I–95) BASE MIX

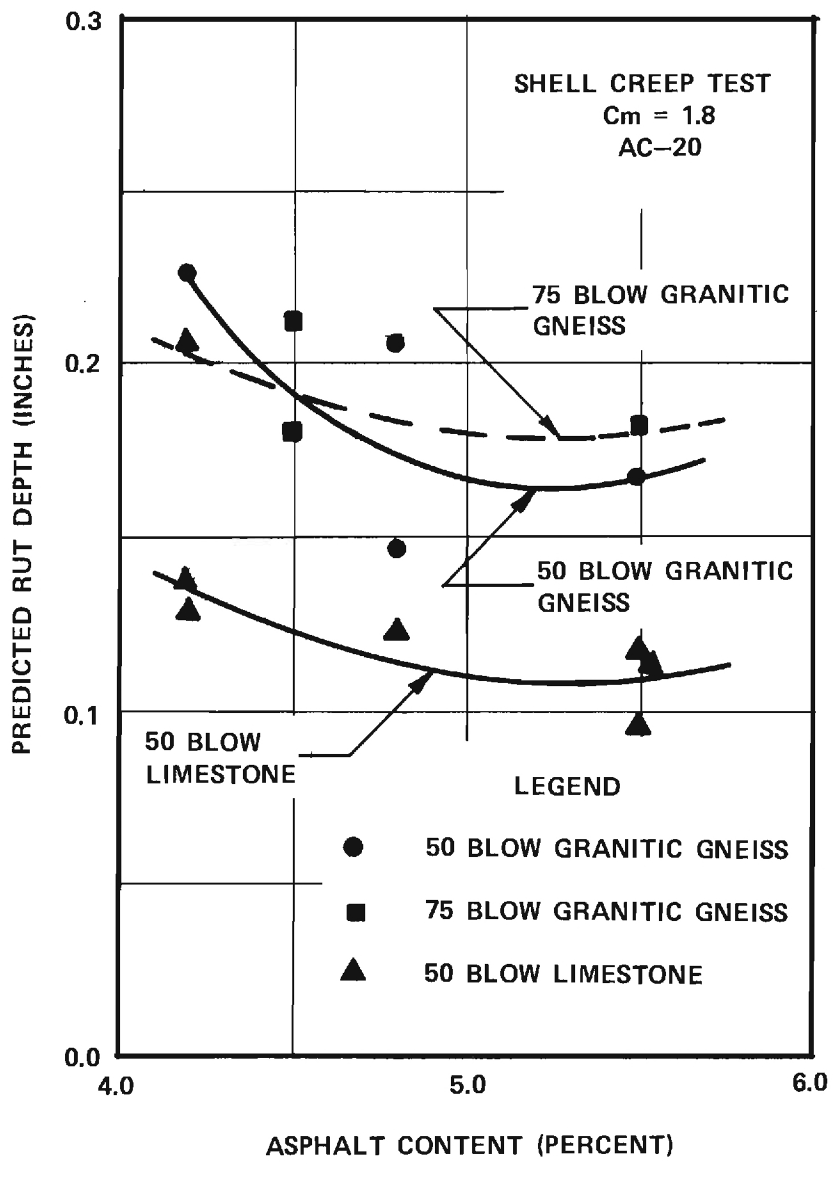


FIGURE 31. VARIATION OF PREDICTED RUT DEPTH IN BLACK BASE WITH ASPHALT CONTENT – SHELL CREEP TEST RESULTS

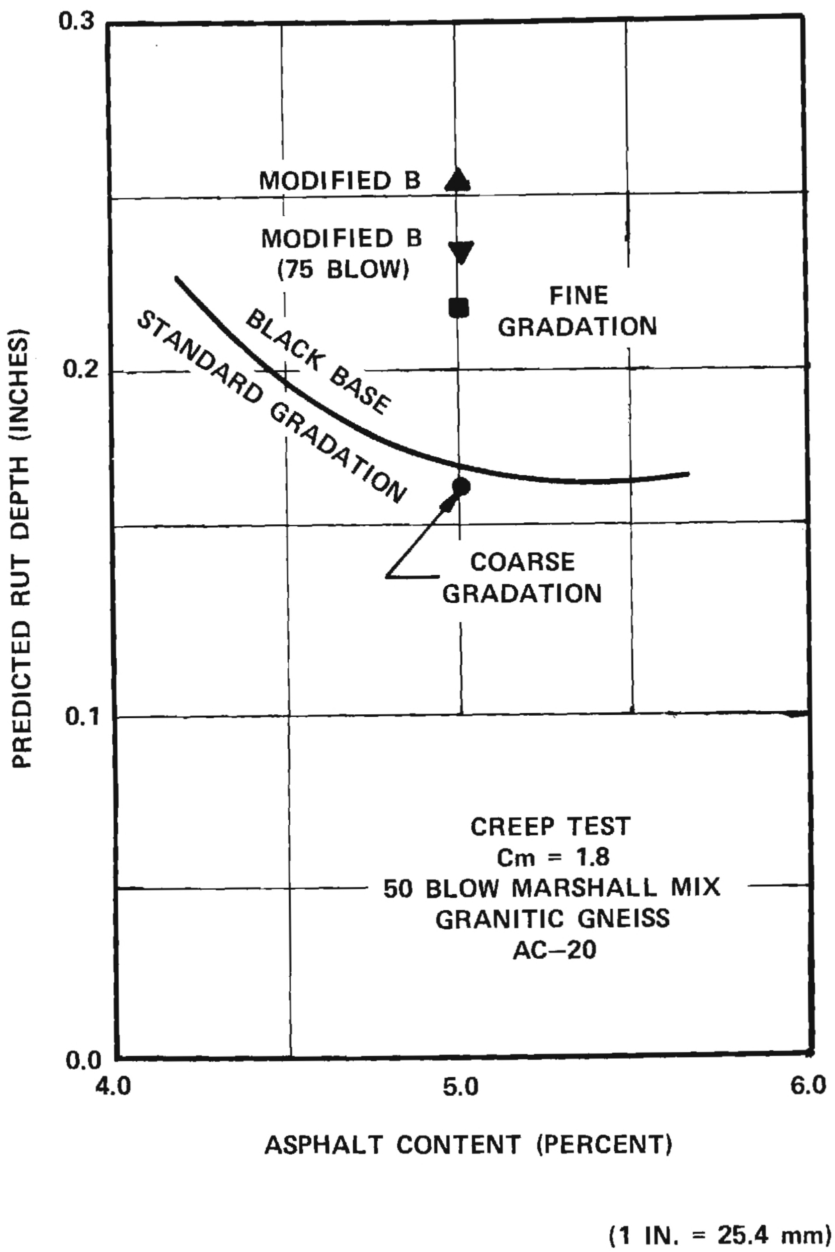
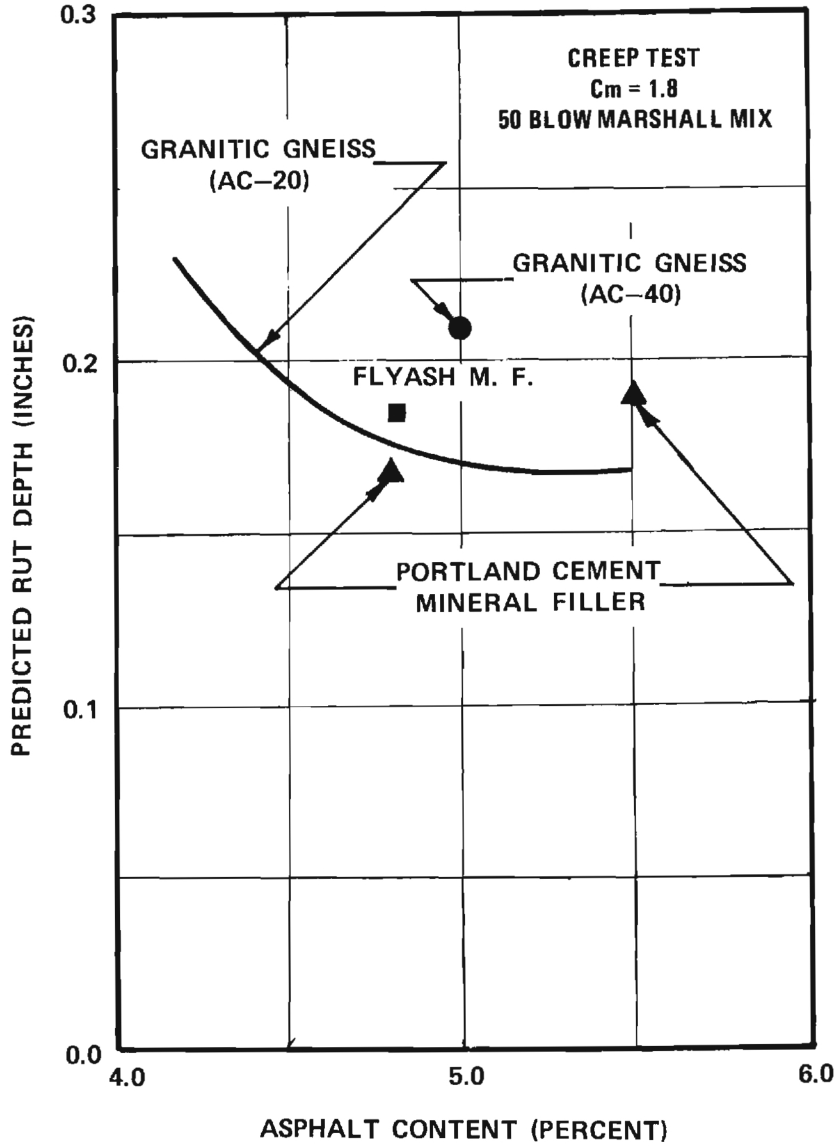


FIGURE 32. EFFECT OF GRADATION AND MIX DESIGN ON PREDICTED RUT DEPTH – SHELL CREEP TEST RESULTS



(1 IN. = 25.4 mm)

FIGURE 33. EFFECT OF MINERAL FILLER AND ASPHALT VISCOSITY ON PREDICTED RUT DEPTH IN BLACK BASE – SHELL CREEP TEST RESULTS

CHAPTER 7

DISCUSSION

Introduction

Numerous field studies [48,49,50-52] have found that hardening of the asphalt concrete surfacing is directly related to surface cracking. A dense graded asphalt concrete mix constructed with a high percent air voids will have a much greater permeability to air, water, and water vapor than a mix having a low amount of air voids. As a result, asphalt concrete surfacings having a high percent voids will undergo hardening primarily due to oxidation of the asphalt cement appreciably more rapidly than a similar mix having a low percent air voids. In addition, laboratory tests [26] show that asphalt concrete mixes having a large percent air voids have low fatigue lives even without a significant amount of hardening occurring.

Potts, Scheweyer, and Smith [49] have concluded that the percent air voids is actually a secondary factor affecting cracking which appears to be controlled by the following primary variables: (1) design percent asphalt; (2) initial asphalt viscosity and temperature susceptibility (these factors appear to affect the coating of the aggregate and the mixing operations); (3) mixing time and temperature; (4) size and grading of the aggregate; and (5) field compaction of the asphalt concrete. Field compaction is greatly influenced by the temperature at the time of rolling, and compaction equipment, ambient temperature conditions, and also long-term effects due to compaction under traffic. During mixing the asphalt may be significantly hardened due to abnormally high temperatures of the asphalt and/or aggregate and also due to excessive mixing times. Higher viscosity asphalts caused by hardening during mixing can result in poor coverage of the aggregate with asphalt and also problems of achieving a low air voids content in the field during compaction. Both Potts et al. [49] and Kandhal and Wenger [51] have concluded that both the viscosity and penetration (at 77°F)[1] of the asphalt after mixing in the pugmill are good indicators of future performance of the asphalt concrete pavement surfacing. The ability to achieve a low void ratio in the field has been found to be related to the asphalt viscosity at 77°F (25°C) after mixing in the pugmill with higher viscosities resulting in higher air voids. Roberts and Gotolski [50] have found that constructing a flexible pavement in the fall rather than in the spring can result in an

1. 77°F = 25°C.

additional 5 to 6 months of pavement life since hardening of the asphalt will not begin to occur until the following spring. Their work has also shown that retained penetration decreases very rapidly as the air voids increase.

The study conducted in Florida by Potts et al. [49] found that sensitive indicators of surface cracking were percent air voids and percent retained penetration. Other factors were also found to be important such as percent asphalt and shear and temperature susceptibility. Cracking was found to occur in the field after nine years in surfacings which had greater than about 4% air voids, viscosities greater than approximately 10 megapoises (77^{o}F)[1], less than 5.5 - 6.0% asphalt content, and retained penetrations (77^{o}F)[1] less than about 35% (Fig. 34). These results tend to agree with those of Simpson et al [53] who found that cracking tended to occur at a viscosity of about 10 megapoises and Zube and Skog [54] who found that a penetration of about 30 corresponded to approximately 10% area cracking.

The fatigue and rutting results presented in Chapter 6 were obtained from laboratory tests performed on compacted specimens. The value of these results are certainly as good as those obtained from other possibly more familiar tests such as the Marshall Mix Design Method. Further, the reliability of these new test methods have at least been partially verified using the results of the trenching studies conducted on I-285. The tests were performed on specimens compacted to the density defined by the Marshall Mix Design Curve. The actual density obtained in the field in many instances would be less than that given on the mix design curve. Also, many factors affect the fatigue characteristics of the asphalt concrete such as construction variables, temperature and moisture gradients, hardening of the asphalt cement, traffic compaction, and oil spilled on the pavement. The test results presented and discussed in this report should give reasonably good comparisons of the relative fatigue and rutting behavior of the various mixes investigated. Also, the rut depths predicted should be within approximately 30 to 50 percent of the values expected to occur in the field although further verification of the methods of rut depth prediction are needed. Nevertheless, because of the numerous factors influencing pavement performance, experience and good engineering judgement should always be used in interpreting and applying the results from the fatigue and rutting tests developed in this study.

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1. $77^{o}F = 25^{o}C$.

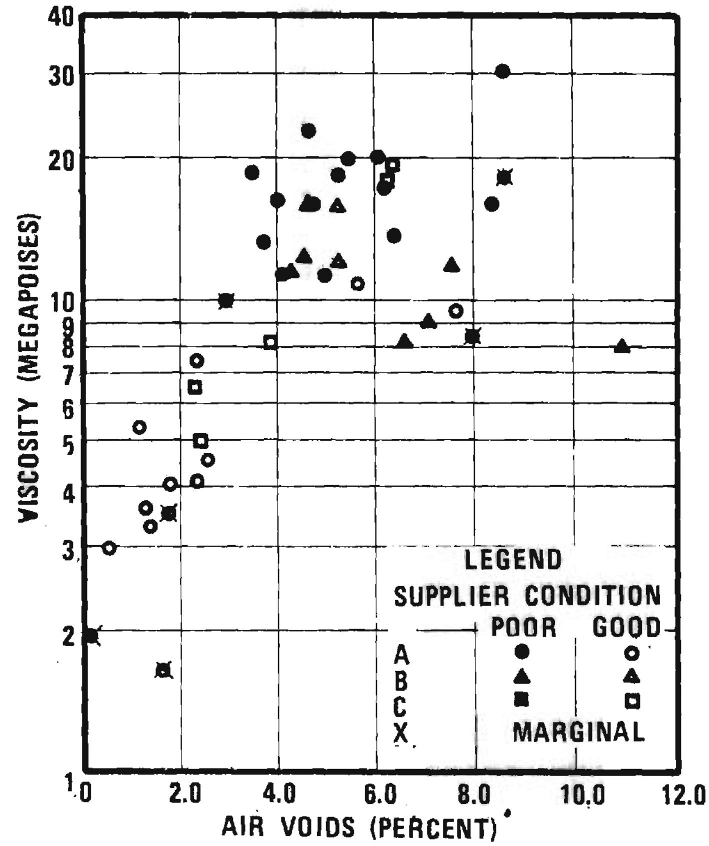

FIGURE 34. EFFECT OF RECOVERED VISCOSITY AND PERCENT AIR VOIDS ON PAVEMENT CRACKING (REF. 49)

Fatigue

Interpretation of Test Results

The interpretation of the laboratory fatigue test results presented in this section is based on both the load method and the elastic theory method. The load method involves determining for different mixes the repetitions required to cause failure by a constant repeated load applied to the beam fatigue specimen. The load method results presented in this section are for a repeated load of 80 lbs. (360 N) and were obtained from the relationships between load and repetitions to failure given in Chapter 6 (Figs. C-1 to C-11).

The elastic theory approach for interpreting fatigue test results involves using a suitable elastic layered theory to calculate the tensile strain in the design pavement structure using the stiffness of the asphalt concrete mix evaluated in the fatigue test. The number of repetitions of loading required to cause failure of the pavement structure under the design loading is then obtained from the experimentally evaluated fatigue curves presented in terms of strain. The calculated tensile strain in the pavement is entered into the fatigue curve and the number of repetitions to failure is determined. The elastic theory approach requires determining the tensile bending strain in the fatigue test specimen. The tensile strain can be determined either directly using a strain gauge as done in this investigation or it can be determined indirectly from the measured centerline deflection of the fatigue specimen. The fatigue test results analyzed by the elastic theory approach in this investigation used a pavement structure consisting of a 10.5 in. (267 mm) structural asphalt concrete layer resting directly on a subgrade. The pavement was subjected to an 18 kip (80 kN) dual wheel axle loading. The subgrade was assumed to have a resilient modulus of 4,000 to 6,000 psi (27,600 to 41,900 kN/m^2). The theoretical relationship between critical bending tensile strain in the asphalt concrete and the bending modulus of elasticity of the asphalt concrete mix is presented in Fig. 35. This relationship was used in the interpretation of the laboratory fatigue test data using the elastic theory approach.

The load method of interpretation of the fatigue test results gives a straightforward, direct comparison of relative fatigue performance. The elastic theory approach involves the use of elastic layered theory which makes several idealized assumptions and uses the measured bending tensile stiffness which tends to be relatively hard to evaluate. For these reasons, the load method of interpretation is favored at this time for evaluating the relative fatigue life of mixes. Further studies might, however, show the elastic

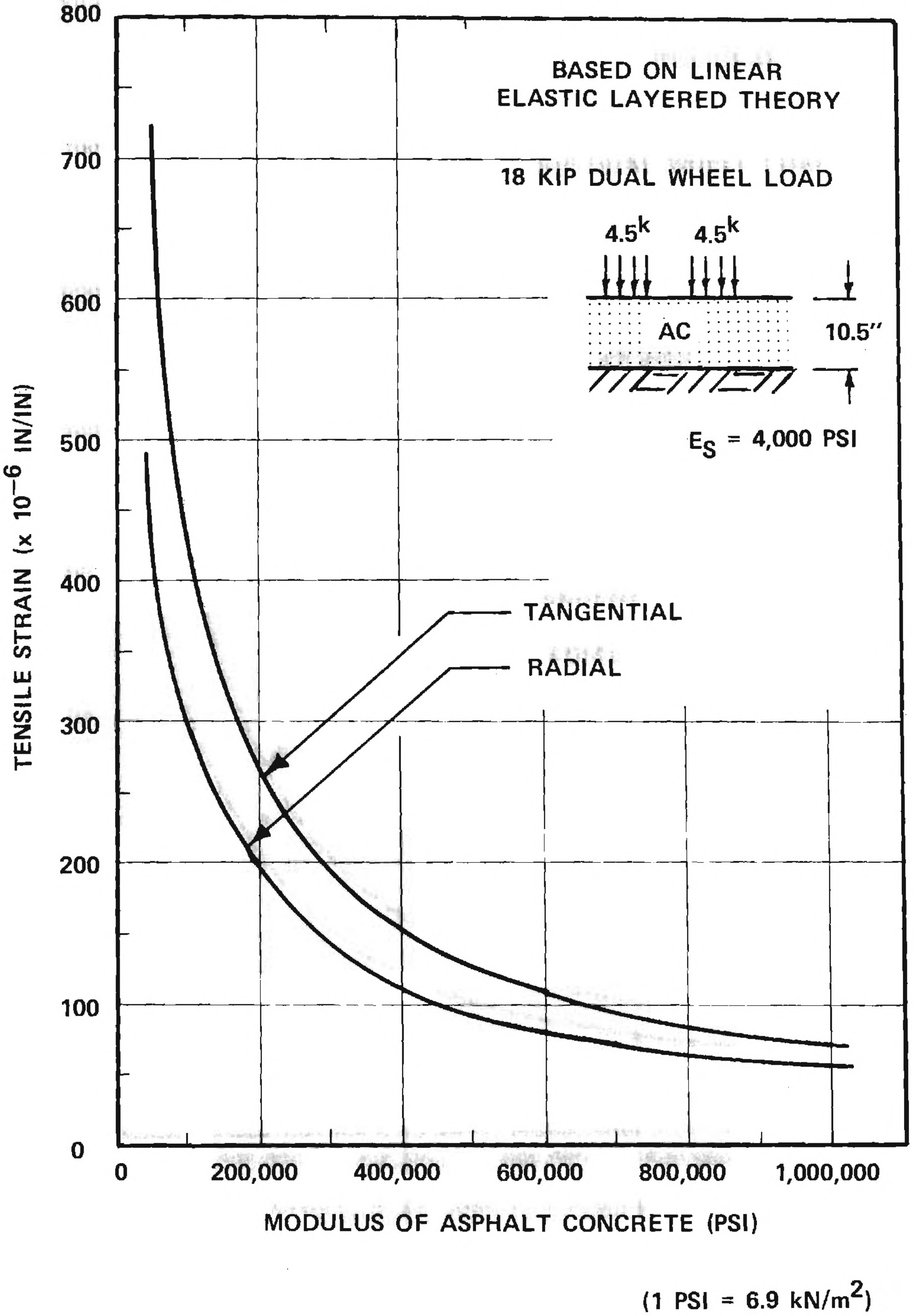

FIGURE 35. TENSILE BENDING STRAIN AS A FUNCTION OF BENDING MODULUS FOR A 10.5 IN. THICK FULL–DEPTH ASPHALT CONCRETE PAVEMENT

theory approach to give better comparisons of in-situ fatigue performance.

Influence of Air Void Content

The air void content and asphalt content of the mix were found from the laboratory test results to be the two independent variables that had a predominant effect on the fatigue life of the mix. As discussed in Chapter 2, the air void and asphalt content has also been found to be the dominant variable by Brown and Pell [45] and by Francken and Verstraeten [28]. Fig. 36 shows that the fatigue life of a mix is very nearly inversely proportional to the air voids on a log-log plot. As a result, in the range of 3 to 8 percent air voids the fatigue life was found from the experimental results to be extremely sensitive to small changes in air voids. For example, decreasing the air voids from 8 to 6 percent increased the fatigue life by a factor of approximately 9. Further, a decrease in air voids from 6 to 3 percent was found to result in an increase in fatigue life by a factor of approximately 200.

The resulting air voids in a mix are influenced by a number of factors including aggregate shape, surface texture and gradation, asphalt viscosity, and compaction level. Since these variables influence the resulting air voids in the mix, they can have an important effect on fatigue performance as discussed subsequently. Determining the actual increase in fatigue life gained from placing in the field a mix at a relatively low initial air void content as compared with a higher one is complicated by the significant beneficial effect of traffic compaction that would occur in the higher void content mix. On the other hand, the mix initially placed at the higher void content would undergo more rapid hardening of the asphalt cement than the mix put down in a more dense state. The effects of traffic compaction on fatigue life are considered in detail in the section on Influence of Marshall Mix Design Procedure.

Influence of Asphalt Content

The test results presented in Figs. 37 through 39 show that a relatively small increase in asphalt content in a 50 blow Marshall Mix black base significantly increases the fatigue life of the mix. The beneficial effect of increasing the asphalt content is particularly large in the range from 4 to 5 percent asphalt. As illustrated in Fig. 39, an increase in asphalt content from 4.25 percent to 4.5 percent was found to increase the fatigue life by 350 percent, and from 4.5 percent to 4.75 percent by 95 percent. Thus, the relationship between fatigue life and asphalt content is one of diminishing returns with increasing asphalt content. Areas of the pavement constructed with an

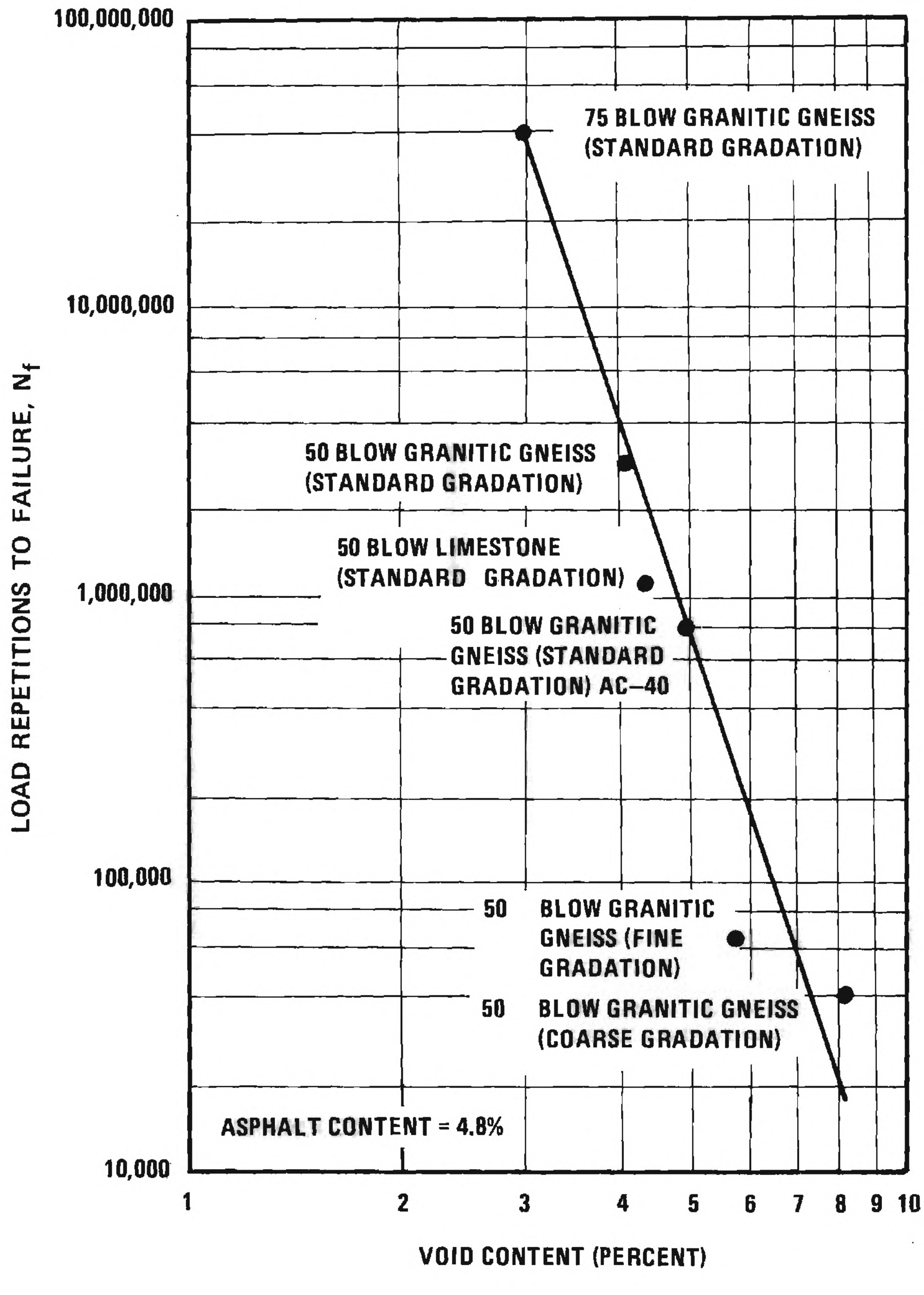

FIGURE 36. EFFECT OF AIR VOIDS ON FATIGUE PERFORMANCE AT AN ASPHALT CONTENT OF 4.8 PERCENT

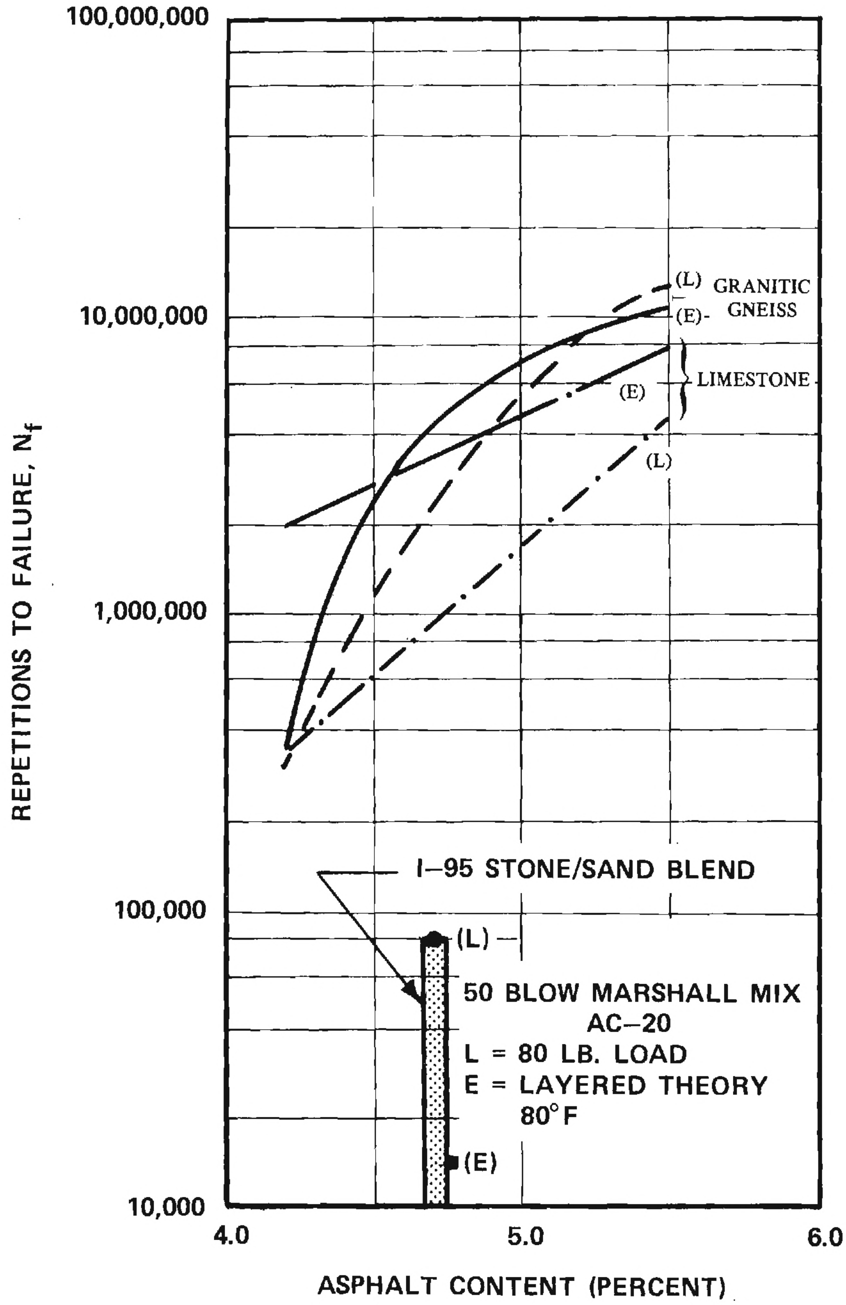

FIGURE 37. EFFECT OF AGGREGATE AND ASPHALT–CEMENT ON FATIGUE LIFE – BLACK BASE MIX

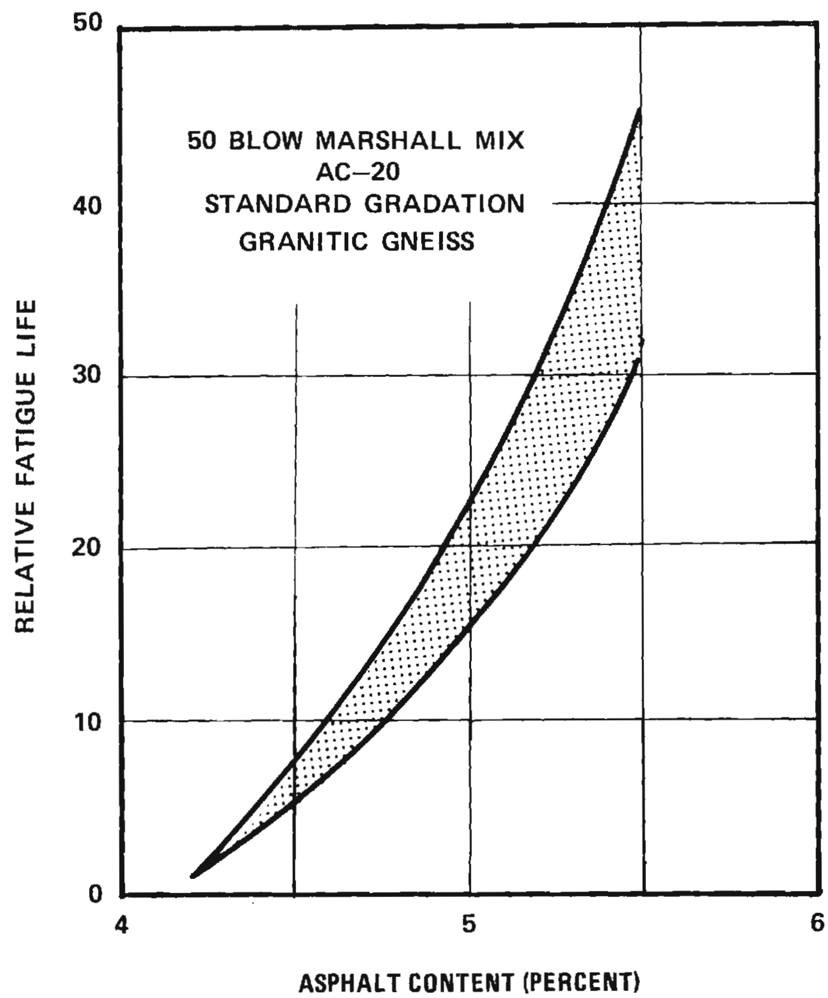

FIGURE 38. EFFECT OF ASPHALT CONTENT ON RELATIVE FATIGUE LIFE – BLACK BASE MIX

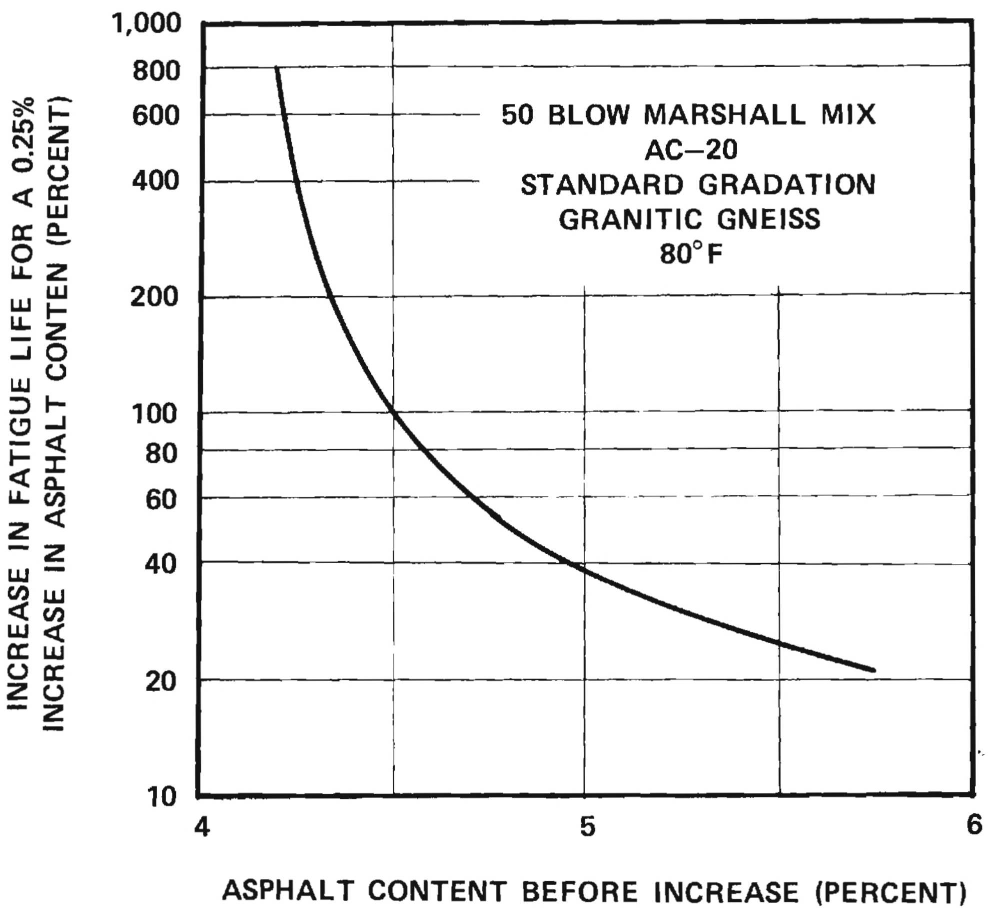

FIGURE 39. EFFECT ON FATIGUE LIFE OF A 0.25 PERCENT INCREASE IN ASPHALT CONTENT – BLACK BASE MIX

asphalt content slightly lower than the design value would be significantly more susceptible to fatigue distress than areas having the design asphalt content. This important finding indicates that an effective approach for increasing the fatigue life of a pavement would be to reduce the variation in asphalt content below the design value by requiring a closer tolerance on the asphalt content. The specifications and construction techniques required to more closely control the asphalt content *may not*, however, be practical.

Other approaches which would also significantly increase fatigue life would be to either increase the design asphalt content of the mix a relatively small amount above the optimum value or define optimum in a different way than is presently done. Although increasing the asphalt content would probably not be as efficient as reducing the variation in asphalt content below the design value (refer to Fig. 39), this technique could be readily implemented.

Influence of Marshall Mix Design Procedure

The fatigue life of a mix can be significantly increased by going from a 50 blow to a 75 blow Marshall Mix as illustrated in Fig. 40 and 41. The beneficial effect on fatigue life, however, rapidly decreases at a decreasing rate going from 4 to 5 percent asphalt content (Fig. 41). For asphalt contents between 5 and 5.5 percent the ratio of fatigue life is almost constant with the fatigue life of the 75 blow mix being approximately four times greater than the 50 blow mix. With time, compaction under traffic will occur in the asphalt concrete placed at a density corresponding to the 50 blow Marshall Design. A smaller amount of compaction under traffic would also be expected to occur in a 75 blow mix. Further, during the period of traffic compaction, more rapid hardening (as reflected by a reduction in penetration) of the asphalt cement in the 50 blow mix compared to the 75 blow mix would also take place due to a higher air void content in the 50 blow mix. These two opposing factors tend to greatly complicate evaluating from the experimental fatigue test results the beneficial effects that would actually be derived in the field in going from the presently used 50 blow Marshall Mix Design to a 75 blow design.

The effect of traffic compaction can be approximately considered by the simple graphical construction shown in Figs. 42 and 43. In these figures the upper line shows the remaining fatigue life of a 75 blow mix with increasing time under traffic loading. The lower family of curves indicates the remaining fatigue life of a corresponding 50 blow mix which undergoes traffic compaction. The family of curves for the 50 blow mix is

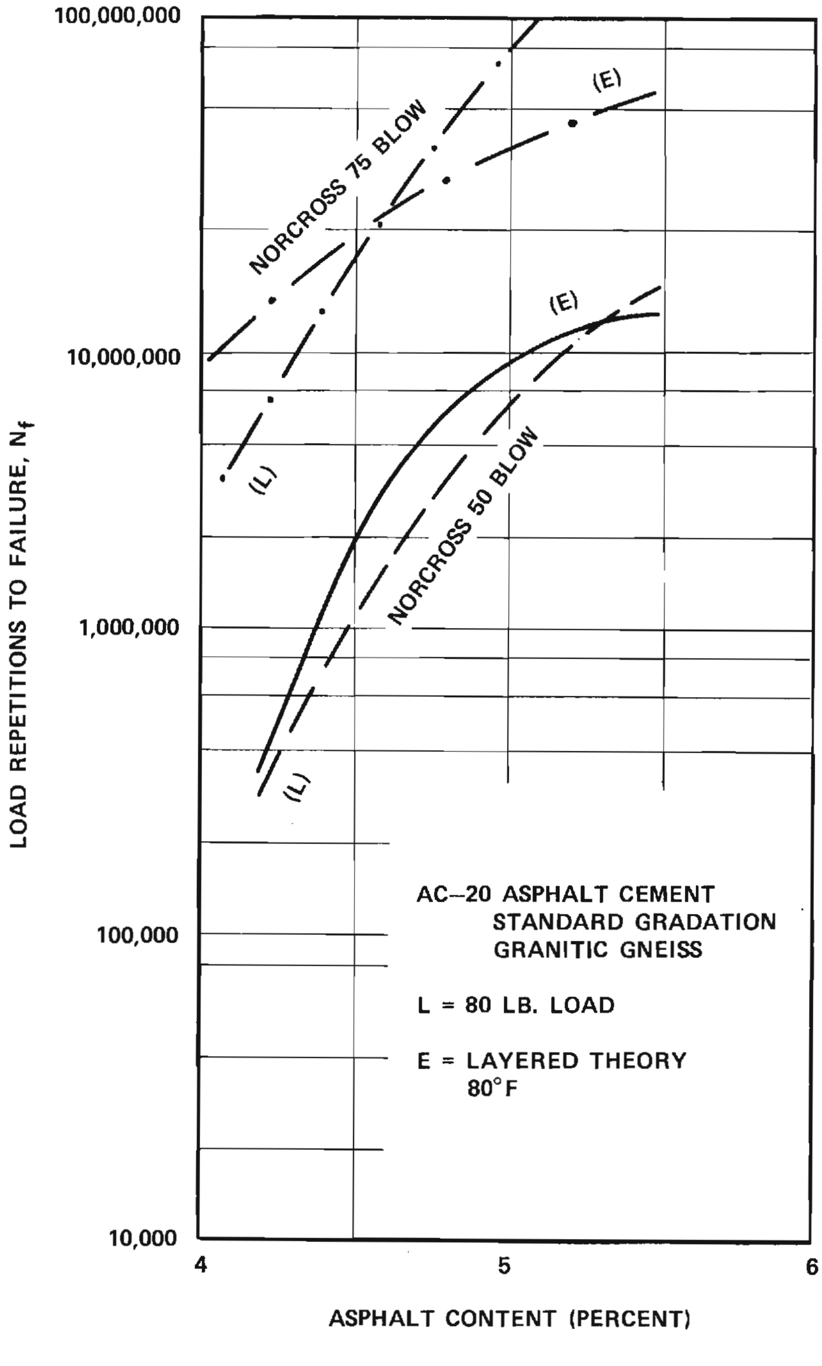

FIGURE 40. COMPARISON OF FATIGUE LIFE OF 50 AND 75 BLOW MARSHALL MIX DESIGNS – BLACK BASE MIX

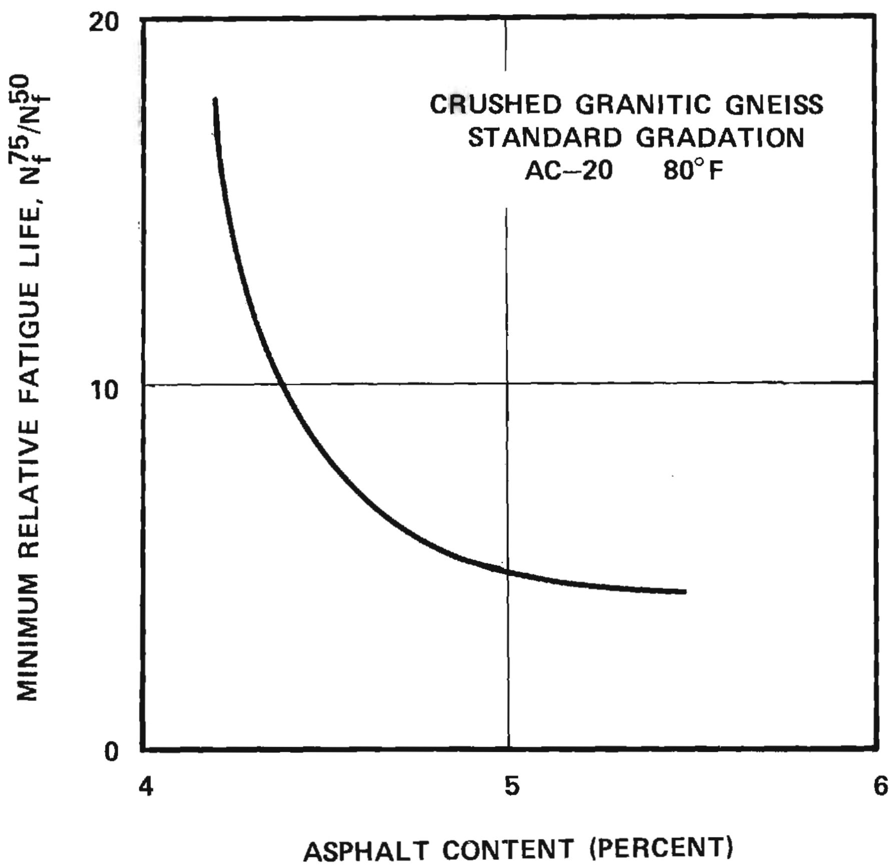

FIGURE 41. EFFECT ON RELATIVE FATIGUE LIFE OF GOING FROM 50 TO 75 BLOW MARSHALL MIX DESIGN – BLACK BASE MIX

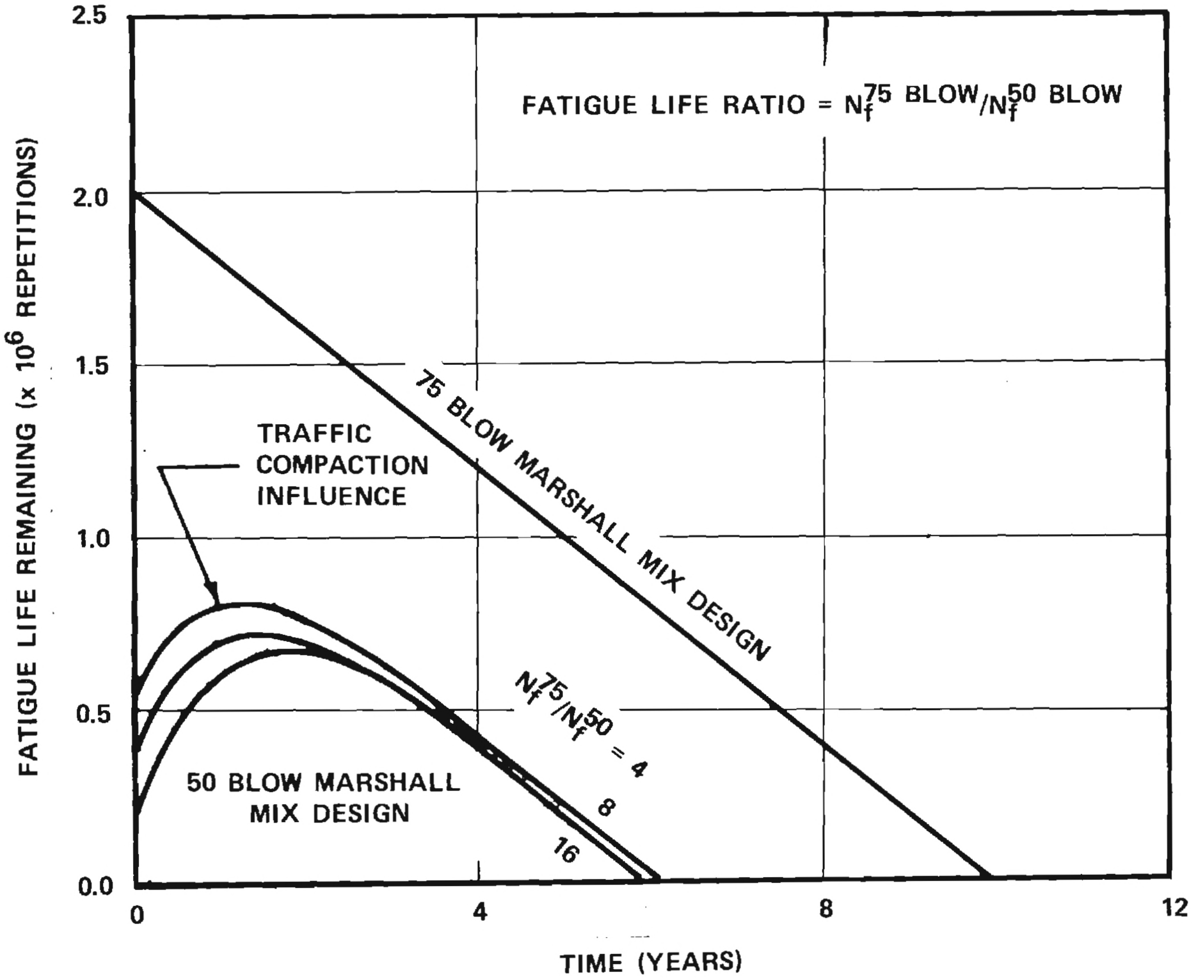

FIGURE 42. EFFECT OF COMPACTION ON 50 BLOW MARSHALL MIX DESIGN ASPHALT CONCRETE – 10 YEAR PAVEMENT LIFE

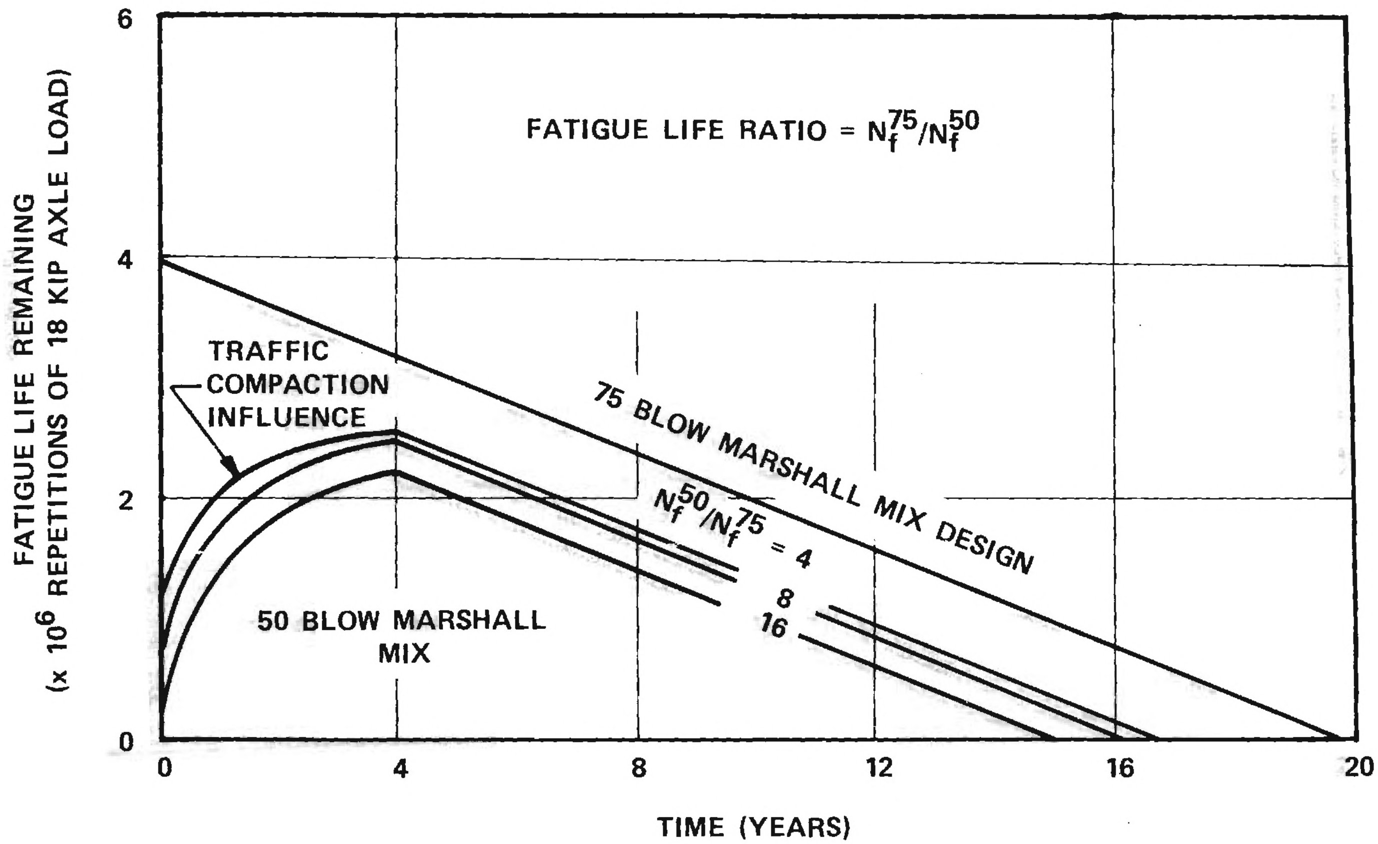

FIGURE 43. EFFECT OF TRAFFIC COMPACTION ON 50 BLOW MARSHALL MIX DESIGN ASPHALT CONCRETE – 20 YEAR PAVEMENT LIFE

for varying fatigue life ratios. The fatigue life ratio is the ratio of the fatigue life of a 75 blow mix to that of a 50 blow mix. The fatigue life ratio determined from the laboratory fatigue tests generally varied from approximately 5 to 8 for asphalt contents going from 5.5 to 4.5 percent. Figures 42 and 43 show that traffic compaction should increase the fatigue life of a 50 blow mix by approximately 300 to 400 percent depending on the actual fatigue life and the fatigue life ratio of the mix. Traffic compaction apparently has a much greater effect on mixes having large fatigue life ratios compared to those with smaller ratios. In going from a 50 to a 75 blow mix, the fatigue life would be increased by approximately 67 percent in a pavement with a 50 blow fatigue life of 6 years (Fig. 42). For a pavement with a 50 blow fatigue life of approximately 15 years (Fig. 43), going to a 75 blow mix would increase the fatigue life by about 25 percent. Therefore, going to a 75 blow mix should be very beneficial for pavements with 50 blow surfacings which presently have relatively short fatigue lives on the order of 6 to 10 years.

The graphical method used to construct the two figures illustrating the effect of traffic compaction used the following somewhat idealized assumptions: (1) The rate of load application is uniform over the design period, (2) fatigue damage is in direct proportion to the applied number of wheel loads (this is suggested by Minor's hypothesis), (3) the rate of loss of fatigue life is the same in both the 50 and 75 blow mixes, and (4) traffic compaction in the 75 blow mix is negligible. Traffic compaction is assumed to occur over a period of 4 years increasing at a decreasing rate with time. Although field measurements [56] indicate that a significant portion of traffic compaction would probably occur in about two years, four years was used in the two examples to at least crudely allow for the (1) detrimental effect of more rapid hardening in the 50 blow mix and (2) the beneficial effect of some traffic compaction occurring in the 75 blow mix.

In at least some instances the actual fatigue life of heavily trafficked pavements in Georgia has been approximately 8 to 10 years. For pavements having fatigue lives on the order of these pavements, going from a 50 blow to a 75 blow Marshall Mix Design procedure should substantially increase the fatigue life of the pavement.

Influence of Asphalt Cement Viscosity

An AC-20 viscosity grade asphalt cement was used as the standard asphalt cement in the fatigue and rutting tests. To determine the effects on fatigue life of using a more viscous asphalt cement, an AC-40 was used for a limited number of tests on the standard gradation 50 blow black base mix. In the usual working range of 4.5 to 5.25 percent asphalt, specimens prepared using the AC-20 asphalt cement had a significantly higher fatigue life than specimens prepared using AC-40 (Fig. 44). Near 4 and 5.5 percent asphalt which was at the limits tested, the AC-20 mix appeared to have a slightly greater fatigue life, but the difference was much less than in the working range of 4.5 to 5.25 percent. One possible explanation of the poorer performance of the AC-40 mix might be that the higher viscosity of this asphalt cement resulted in poorer coverage of the aggregate [49]. The laboratory fatigue test results indicate that for the mix investigated use of an AC-20 asphalt cement is more desirable from the standpoint of fatigue than an AC-40. Field experience also indicates that use of a higher viscosity asphalt cement results in a shorter fatigue life [49].

Influence of Mineral Filler and Gradation

The influence of portland cement filler, flyash filler, and also coarse and fine gradations on fatigue life is summarized on Fig. 45. The mixes having the portland cement and flyash filler had the same gradation as the standard granitic gneiss mix which is shown on the figure with solid and dashed lines. To study the influence of filler type, the crushed stone mineral filler passing the No. 200 sieve used in the standard granitic gneiss mix was replaced with portland cement or flyash.

The laboratory fatigue test results indicate that use of flyash filler will result in a mix having a smaller fatigue life than if crushed mineral aggregate is used. If portland cement filler is used, the fatigue test results interpreted using the load method indicate a slight reduction in fatigue life. However, if the elasticity approach is used to interpret the data, the fatigue life of this mix appears to be increased. These test results therefore indicate that the fatigue life using portland cement filler should be about the same as mineral aggregate and might even be increased.

Both the coarse and fine gradation mixes were found to have significantly lower fatigue lives than the standard mix. This important difference in fatigue life is primarily due to the standard mix having a lower void content than either the coarse or fine mixes. The important effect of air void content on fatigue performance has been previously illustrated in Fig. 36.

Estimation of Rut Depth

Both the repeated load and creep test results were used to predict the rutting characteristics of the base course mixes investigated. Based on results of extensive tests, the tentative conclusion is made that the repeated load triaxial test appears to

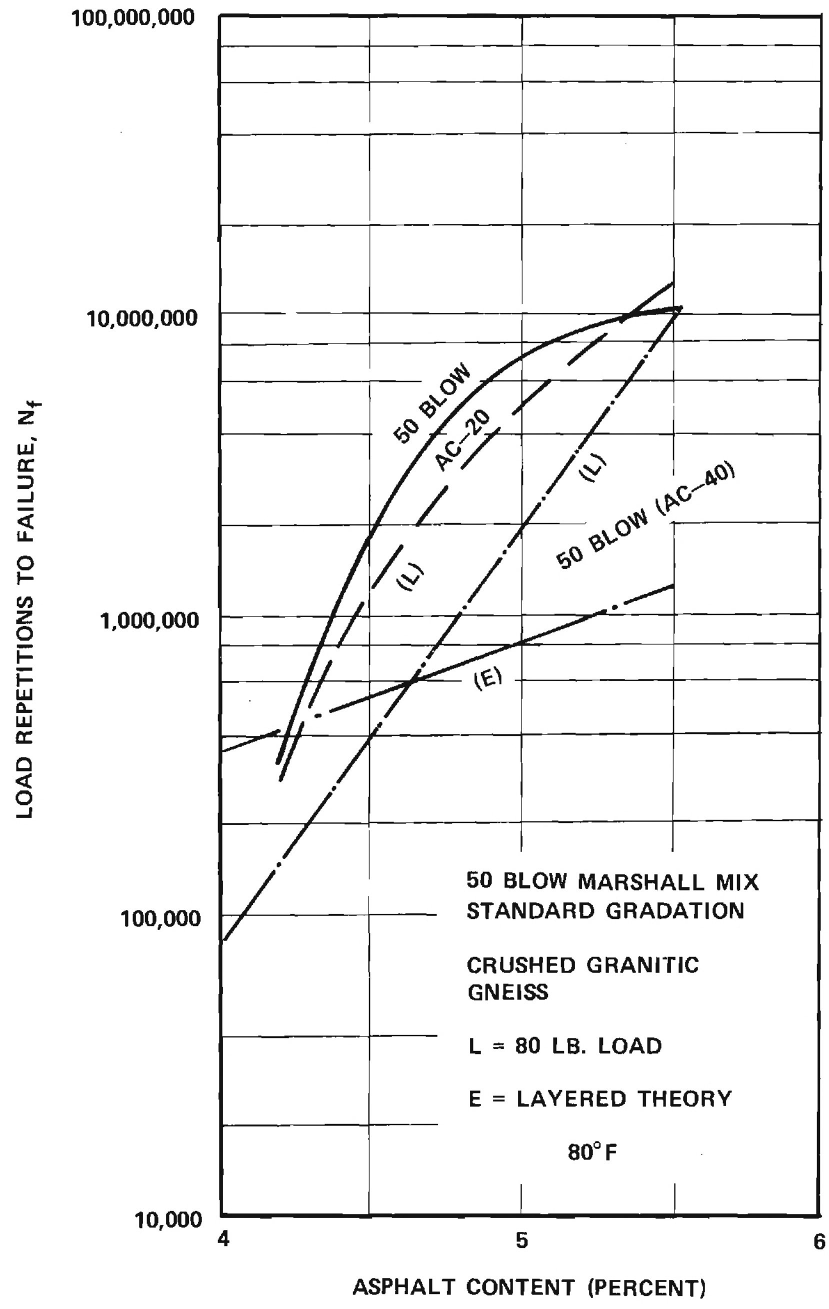

FIGURE 44. COMPARISON OF FATIGUE LIFE USING AC–20 AND AC–40 ASPHALT CEMENT – BLACK BASE MIX

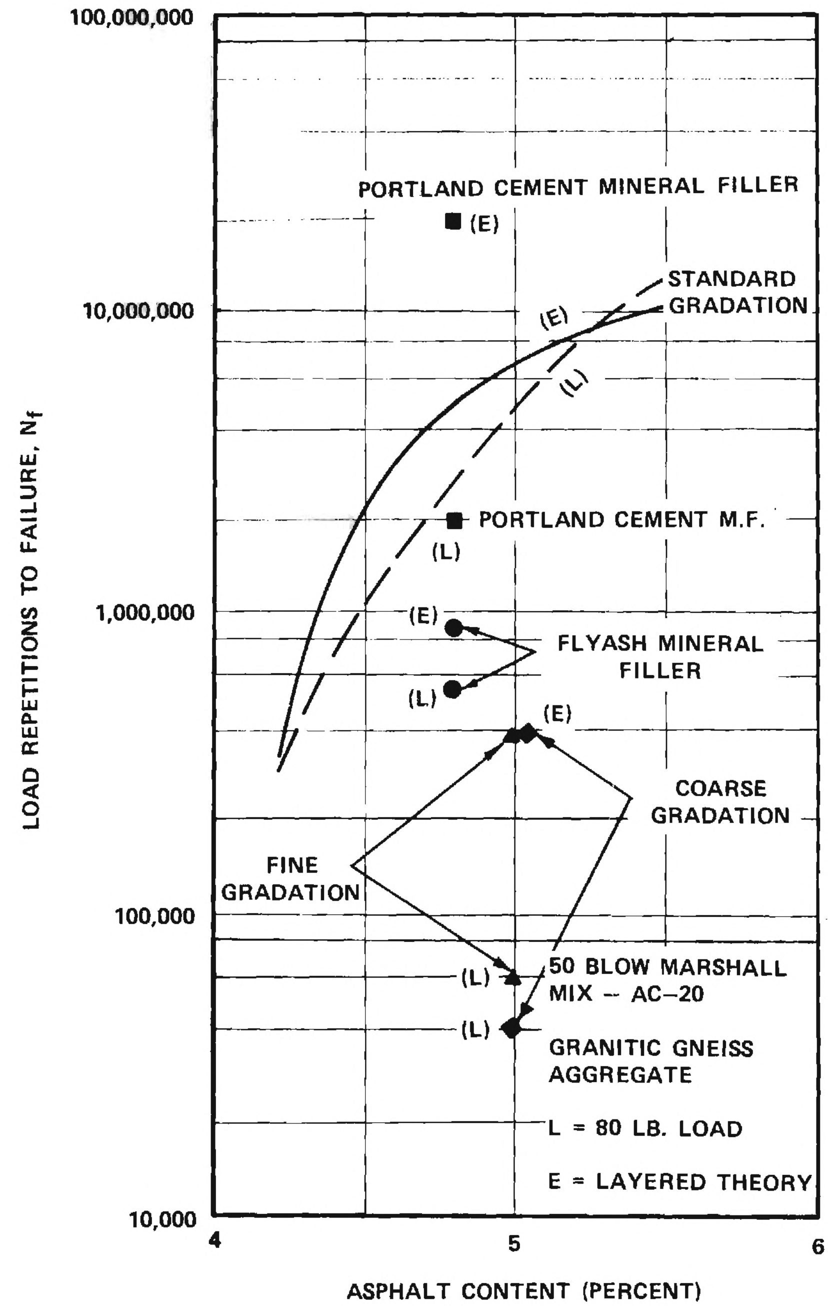

FIGURE 45. INFLUENCE OF MINERAL FILLER AND GRADATION ON FATIGUE LIFE – BLACK BASE MIX

give a better comparison of rutting characteristics than the creep test interpreted using the Shell Method. Additional comparisons of predicted rut depths with those measured in the field should be conducted to verify which method is more appropriate. The reasons for preferring the repeated load triaxial test method over the Shell creep test are as follows:

1. As shown by Fig. 46, the predicted rut depths from the triaxial test results were found to gradually increase for asphalt contents greater than 4 percent. Rut depths predicted from the Shell creep test, however, indicated a very small decrease up to an asphalt content of approximately 5.5 percent. The repeated load test method for predicting rut depth therefore gives more conservative results since it indicates the rut depth should increase with increasing asphalt content. Further, it is reasonable to believe that at asphalt contents greater than about 5 percent rut depth would increase with increasing asphalt content.
2. The stone-sand specimens (I-95 mix) failed in the Shell creep test at the conventionally used stress level of 15 psi (103 kN/m^2). The effective vertical stress for use in unconfined testing would be slightly greater than twice this value. None of the stone-sand specimens tested in the confined triaxial test failed although the maximum permanent deformation was almost twice as great as the average value observed for the standard granitic gneiss mix. The mix would not be expected to actually fail when subjected to repeated traffic loadings in the field, but rather undergo large permanent deformations. The unconfined Shell creep test as performed in this investigation therefore appears to give questionable results. Further, the repeated load test results indicate that a small amount of confinement can significantly reduce the permanent deformation occurring in at least some mixes.
3. The repeated load tests performed on the AC-40 asphalt cement indicated slightly less rutting than for an AC-20 mix; predicted rut depths from Shell creep test results indicated slightly greater rut depths. In both cases, however, the difference in rut depth was not great.
4. Both test methods indicated the limestone mix would rut less than the granitic gneiss. However, the percent difference in rut depths predicted using creep test results was generally almost twice as great as for the

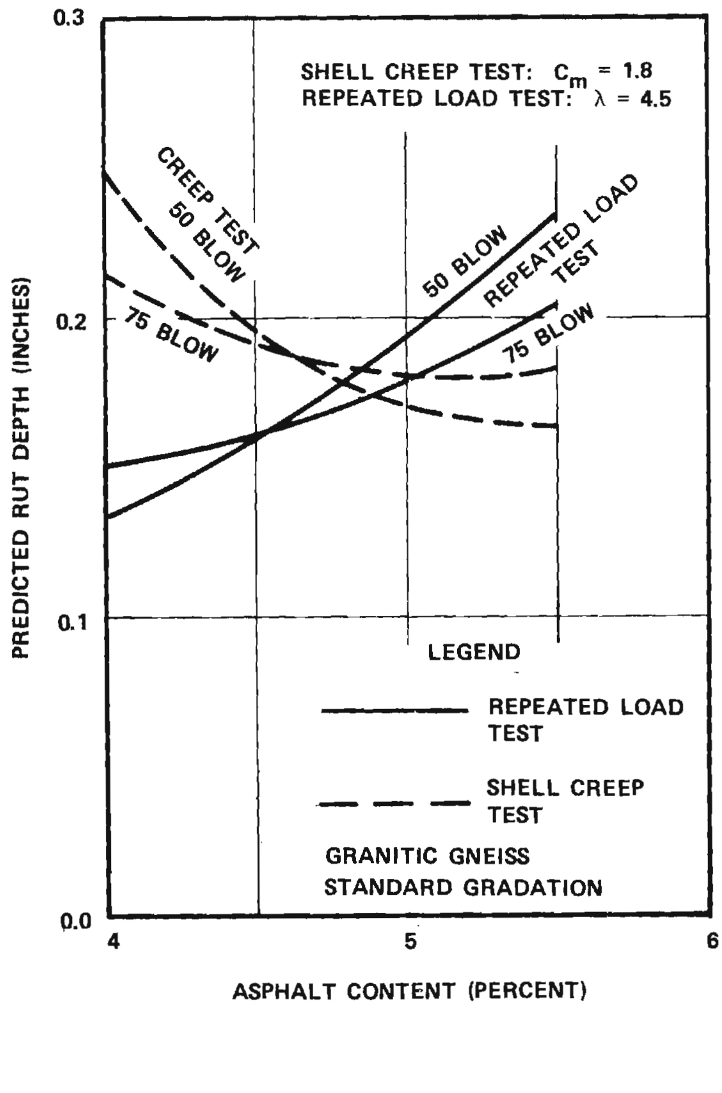

FIGURE 46. COMPARISON OF RUT DEPTH PREDICTIONS FROM REPEATED LOAD AND CREEP TESTS – BLACK BASE MIX

triaxial test.

Rut depths calculated from the repeated load triaxial tests were predicted using the following simplified engineering approach:

$$\Delta H = \lambda \, \varepsilon_p H \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (12)$$

where ΔH = predicted rut depth

H = thicknesses of the asphalt concrete layer. A limiting asphalt concrete layer thickness of 10.5 in.(267 mm) should be used for normal wheel loadings

ε_p = average permanent strain occurring in the layer after 100,000 load repetitions as measured by the repeated load triaxial test

λ = correction factor that considers the test was performed to only 100,000 repetitions and also includes the correlation between the laboratory test and field performance.

It is desirable that the repeated load triaxial test be performed at the appropriate vertical stress level given by $Z\sigma_o$, equation (11) using the effective confining pressure in the layer. The actual distribution of rutting through an asphalt concrete pavement layer has not yet been clearly defined. On a small scale test track, however, Hofstra and Klomp [31] found that rutting was either almost constant with depth or decreased near the bottom of the layer. Further, several field studies [30] have indicated a limiting depth exists beyond which significant amounts of rutting in the asphalt concrete does not occur. The AASHO Road Test results indicate that more rutting occurs in an asphalt concrete surfacing 3-1/2 to 4-1/2 in. (89 to 114 mm) in thickness than in an unstabilized granular base 6 in. (152 mm) thick located beneath the asphalt concrete layer. This field evidence suggests that more rutting should occur in the upper part of an asphalt concrete layer than in the lower portion.

From the above discussion, the distribution of rutting with depth shown in Fig. 47 is proposed for a preliminary estimate. From the definition of Z given in Chapter 3, p. 24 , the average vertical stress in the layer which causes deformation is equal to $Z\sigma_o$ where σ_o is the tire inflation pressure. For spring and summer pavement temperatures existing in Georgia, the average theoretical value of Z for the pavement structure investigated is approximately 0.4. Now assume that the rut depth is proportional to the Z function (which is a good first approximation), and apply this assumption to the distribution of rutting with depth shown in Fig. 47. For these conditions the Z function

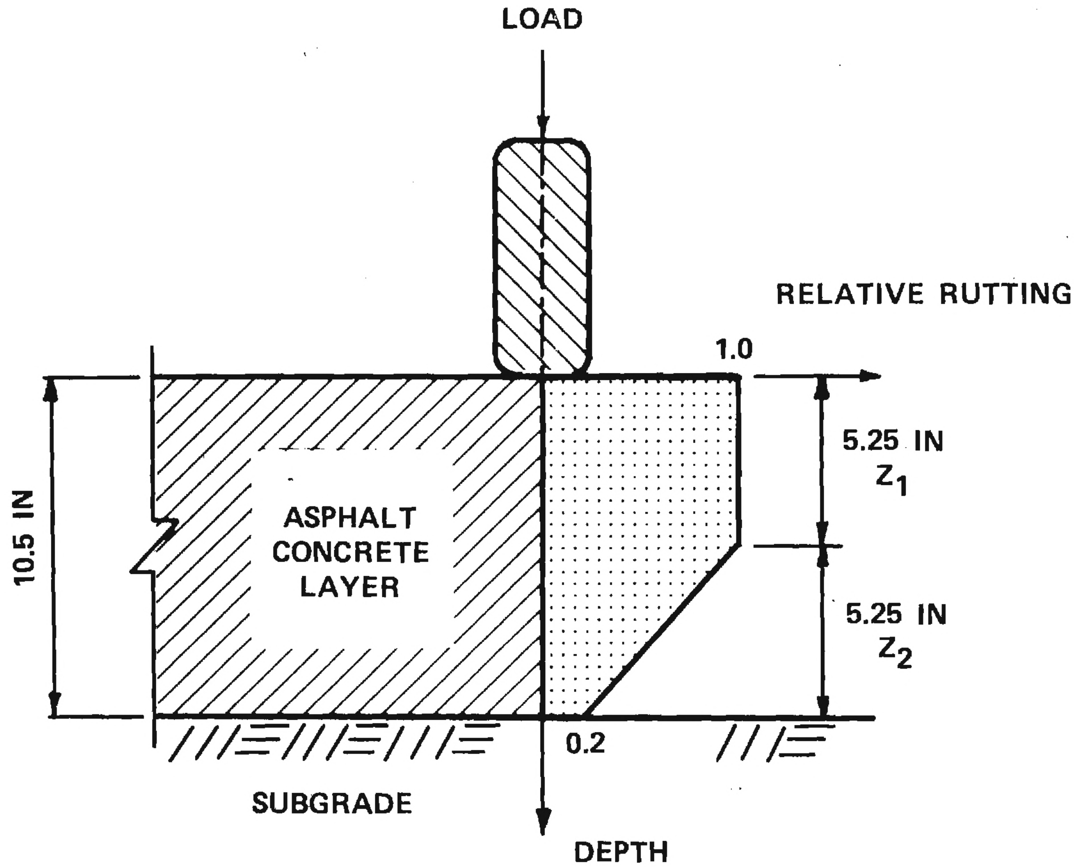

FIGURE 47. APPROXIMATE VARIATION OF RUTTING WITH DEPTH IN A THICK ASPHALT CONCRETE LAYER

for the upper half of the layer is equal to 0.50 and for the lower half the average value is 0.30. The average vertical stress for thick asphalt concrete layers approximately 10.5 in. (267 mm) or more in thickness is therefore approximately equal to the average Z function for the layer which is 0.4 multiplied by the tire contact pressure. For the upper approximately 5.25 inches (130 mm) of a thick asphalt concrete layer, the average vertical stress from this development is approximately equal to the average Z factor in the upper half of the layer (0.50) multiplied by the tire contact pressure. Of course, a uniform Z factor of 0.40 could be used throughout the layer as a preliminary approximation.

For a summer tire inflation pressure, σ_o of 85 psi (590 kN/m^2), the average vertical stress in a relatively thick asphalt concrete layer is therefore equal to $Z\sigma_o$ which gives 34 psi (230 kN/m^2) for a Z function of 0.4. By correlating measured rut depths with the repeated load triaxial test results, the average confining pressure in the asphalt concrete layer appears to be approximately 3.0 psi (21 kN/m^2). A typical Georgia asphalt concrete base course specimen tested at this confining pressure to 1,000,000 or more repetitions at a deviator stress of 34 psi (230 kN/m^2) would give approximately 0.017 in./in. of permanent strain. For repeated load triaxial tests performed at 95^{o}F (35^{o}C) to 100,000 repetitions using this stress state, λ would be approximately 1.7.

For a pavement thickness of 10.5 in. (270 mm) this gives from equation (12) a rut depth of 0.18 in. (4.6 mm). This rut depth was approximately the value observed to exist on I-285, I-75 and I-85 in Georgia. An asphalt concrete thickness of 10.5 in. (270 mm) is approximately the limiting asphalt concrete pavement thickness beyond which little additional rutting should occur if the pavement is placed on a reasonably good subgrade. This approach assumes all of the rutting to occur in the asphalt concrete layer for thicknesses greater than 10.5 in. (270 mm) and hence tends to be on the conservative side in this respect.

The repeated load triaxial test results presented in this investigation and used to predict rut depth were performed at a confining pressure of 5 psi (34 kN/m^2) and a deviator stress of 25 psi (170 kN/m^2). The temperature was held constant at 95^{o}F (35^{o}C) which is approximately the mean pavement temperature at which rutting occurs in Georgia. For these stress conditions, the value of λ in equation (12) was found to be 4.5 assuming a limiting asphalt concrete thickness of 10.5 in. (270 mm) and a rut depth of 0.18 in. (4.6 mm) [refer to Fig. 48].

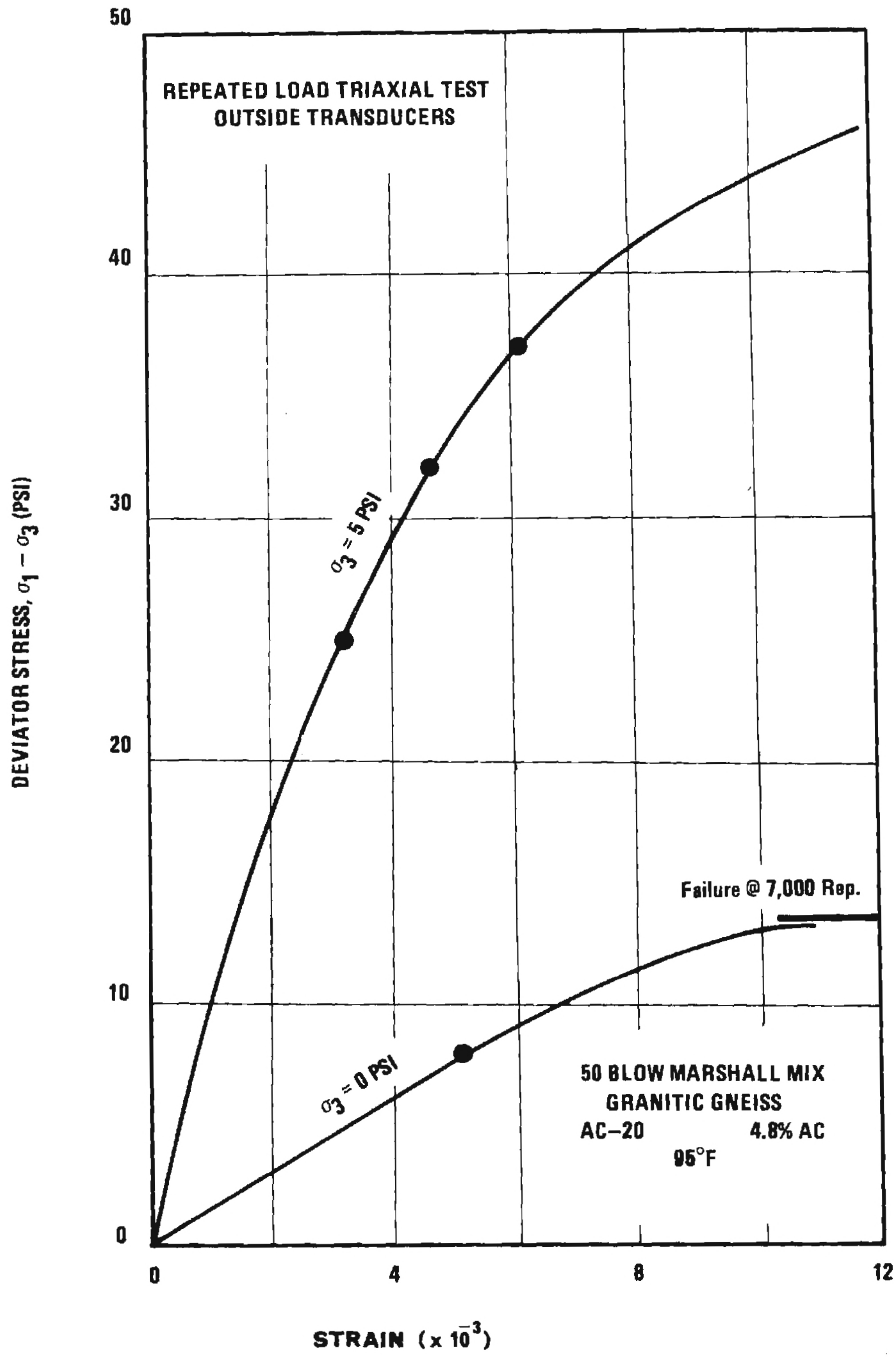

FIGURE 48. VARIATION OF PERMANENT STRAIN WITH DEVIATOR STRESS AND CONFINING STRESS – REPEATED LOAD TRIAXIAL TEST RESULTS

Rut depths from the creep test results were predicted using the Shell approach [40]. This method is completely described in Appendix A, and an example is given to illustrate rut depth computations from creep test results. Based on a correlation with the measured rut depths in Georgia, a C_m factor of 1.8 was used in equation (4) for rut depth computations by the Shell Method.

Rutting

The results of both the creep and repeated load triaxial tests indicate that although rut depth is related to asphalt content, changes in the asphalt content on the order of 1/2 percent from the Marshall Mix Design optimum as reported by the Georgia DOT will not result in a drastic change in rut depth (Fig. 46). For both the 50 and 75 blow granitic gneiss mixes, the Shell creep test indicated that the minimum rut depth would occur in the vicinity of an asphalt content of 5 to 5.5 percent. The repeated load triaxial test results indicated that rut depth would gradually increase from an asphalt content of 4 up to 5.5 percent. Minimum rut depth apparently occurred at an asphalt content less than 4 percent in the repeated load triaxial test. Increasing from a 50 to a 75 blow Marshall Mix Design did not have a significant effect in reducing rutting using either test method.

Use of a crushed limestone aggregate was found from both the repeated load triaxial test (Fig. 26) and the creep test (Fig. 31) to result in less rutting potential than the granitic gneiss tested having the same gradation. The repeated load triaxial test, however, indicates less difference with respect to rutting of the two aggregates than the creep test. The granitic gneiss mixes having a fine and coarse gradation appeared to have essentially the same rutting potential for equal asphalt contents as did the mix having the standard gradation (Fig. 27 and 32). The mix with the fine gradation might be slightly more susceptible to rutting as indicated by the creep test results (Fig. 32).

Both the 50 and 75 blow Modified B mix were found in the Shell creep test at the same asphalt content to be slightly more susceptible to rutting than the black base mix tested having the standard gradation. Use of flyash and portland cement mineral filler did not significantly change the rutting characteristics of the standard granitic gneiss mix (Figs. 28 and 33).

Use of an AC-40 viscosity grade asphalt cement appeared to give the minimum rut potential at an asphalt content in the vicinity of 4.5 percent although more test data is needed to verify this. For the corresponding AC-20 mix the optimum asphalt content appears to be approximately 1/2 to 1 percent less than for the AC-40 mix. Further, the repeated load triaxial test results indicated the AC-40 asphalt cement should give slightly less rutting between 4-1/2 and 5-1/2 percent asphalt. The Shell creep test results indicated that the AC-40 mix would result in slightly greater rut depths. Both test methods, however, indicated that the use of an AC-40 asphalt cement should not have a significant effect on rut depth. Field measurements over a 10-year period of rut depths have been performed by Kandhal and Wenger [51] for asphalt viscosities varying from 966 to 2649 poises. Rut depths were found to be between 0.18 and 0.21 in. (4.6 and 5.3 mm) for all mixes which supports the finding that asphalt viscosity should not have a significant effect on rut depth.

The I-95 aggregate-sand blend was found to be considerably more susceptible to rutting than the standard granitic gneiss mix. Creep test specimens subjected to the conventional 15 psi (100 kN/m^2) unconfined compressive stress failed after approximately 1,000 to 3,000 sec. of loading. On the other hand, repeated load triaxial test specimens did not fail when subjected to a deviator stress of 25 psi (170 kN/m^2) at a confining pressure of 5 psi (34 kN/m^2). The repeated load triaxial test results, presented in Fig. 30, indicate that the rutting potential for a confining pressure of 5 psi (34 kN/m^2) on the average should be approximately 50 percent greater than the standard granitic gneiss mix tested. The poorer performing I-95 materials should rut as much as 75 percent or more than the standard mixes. Based on the results of the creep and repeated load test results, the I-95 mix would be expected to have rut depths ranging from approximately 0.25 to 0.5 inches (6 to 13 mm) with the average value probably being in the vicinity of 0.3 to 0.4 in. (8 to 10 mm).

Mix Design Procedure

The mix design procedure presented in this section is recommended for the design of asphalt concrete base course mixes by the Georgia Department of Transportation. This procedure places more emphasis on designing for fatigue resistance than does the Marshall Mix Design Method as presently applied which considers the rutting (stability) characteristics of the mix but does not maximize fatigue life. The proposed procedure which includes the Marshall Mix Design as the first step is as follows:

1. Marshall Mix Design: The first step in the mix design procedure is to perform a conventional Marshall Mix Design for the material under consideration. A mix should be developed having an air void content less than 6 percent and preferably in the range of 3 to 4 percent. To prevent bleeding of the mix the void content should be greater than 2 percent.

2. Select Design Asphalt Content: Using the Marshall Mix Design results, a design asphalt content should then be selected as high as practical to maximize fatigue life. Increasing the asphalt content 0.25 to 0.5 percent above the optimum value as presently defined by the Georgia DOT should increase the fatigue life as much as 50 to 100 percent or even more compared to the fatigue life that would be obtained at the presently used optimum. The effect of varying asphalt content and air void content can be estimated from Figs. 36, 38 and 39 for most mixes. Therefore for most routine mixes fatigue tests do not have to be performed.
3. Evaluate Rut Potential: Next evaluate the rutting potential of the design mix if this distress mode is considered to be a possible problem. This investigation has shown that rut depth is not highly sensitive to asphalt content and other test variables. Therefore, the repeated load triaxial test need not be performed on most mixes. Conditions where the repeated load test should be performed include base course mixes having design asphalt contents greater than 5.25 to 5.5 percent, mixes using marginal materials and in some instances, mixes having fine gradations.

   A limiting predicted rut depth of 0.25 in. (6 mm) is recommended at this time as a design criteria for conventional mixes. For mixes utilizing marginal materials such as sand-asphalt and stone-sand blends, a higher limiting value of rut depth should in general be used. A preliminary limiting design value of 0.35 to 0.4 in. (9 to 10 mm) would appear suitable for marginal materials. The design cross-slope of the pavement should be considered in establishing the maximum allowable rut depth so that ponding of water does not become a safety problem. Further verification is needed of the rutting criteria proposed for both type mixes.

   The rut depth should be calculated as described in the previous section from the results of repeated load triaxial tests. Preferably two repeated tests should be performed on similar specimens at the trial design asphalt content, and the average predicted rut depth compared with the applicable design criteria. As a simplified alternate, the creep test results could be used to predict the rut depth of the mix. If this less desirable approach is used, a C_m factor of 2.2 is recommended for calculating the rut depth.
4. Evaluate Fatigue Life: Generally the fatigue performance of the mix can be

controlled by adjusting the asphalt content and air voids in the mix to give the required fatigue life. Figures 36 and 39 can be used as a guide in estimating relative fatigue life. The fatigue life of mixes using marginal materials having unknown performance characteristics should however probably be evaluated until sufficient experience with these materials has been gained. In some instances, fatigue tests may also be required on mixes having low asphalt contents or high air voids. Mixes having coarse gradations and at least some mixes with finer gradations are likely to have high air voids and hence have reduced fatigue lives.

The same fatigue test developed in this investigation should be used. Preferably, two identical specimens should be tested at the trial design asphalt content. The fatigue test should be performed at a load of 110 lbs., (490 N) and the average number of repetitions to failure used as the fatigue life of the mix. Comparisons of fatigue mix performance should be made using the constant load approach (i.e., the fatigue life is taken as the number of repetitions of the 110 lb. (490 N) load required to cause failure). Determining the fatigue life of the mix using the elastic theory approach would also be desirable but certainly not necessary. This method, which has been described in the section on evaluating fatigue test results, requires determining the tensile bending strain of the specimen during the fatigue test. As a result this approach is more complicated and appears not to be any better than the load method and possibly is not as good. Therefore, it is recommended that the elastic theory method not be introduced until after the fatigue test has become routine. To be acceptable, the mix should be able to withstand 175,000 to 200,000 repetitions of a 110 lb. (490 N) load.

General Discussion

The test results indicate that the fatigue life of a mix can be greatly influenced by most mix variables such as asphalt content, number of blows used in the Marshall Mix Design, mineral filler, aggregate type, and gradation. In contrast, reasonable changes in these mix variables do not have a very great effect on the rutting potential of the mix. The overall effect on fatigue life of moderate changes in the mix design variables is generally one order of magnitude or more greater than the effects on the rutting potential.

The Marshall Mix Design Method has been successfully used for many years. This method, however, is more closely related to rutting rather than fatigue performance.

In Georgia and throughout the United States, mix designs using the Marshall and Hveem Methods usually result in total pavement rut depths of about 0.20 in. (5 mm)[6], and often the rut depth is even smaller. As a result rutting in general is not an important existing problem, although care should always be exercised to control the rut depth to an acceptable level. Since fatigue failure is the predominant existing distress mechanism in pavements, it is reasonable that the mix design place more emphasis than the Marshall Method on optimizing fatigue life, while at the same time limiting rutting to an acceptable level.

The fatigue life of a mix can be improved by (1) decreasing the initial air voids in the mix by either going to a higher blow mix design or by specifying a higher percentage of compaction of the mix in the field, (2) increasing the minimum asphalt content of the mix as placed in the field and (3) controlling the materials and gradation of the materials used in the mix. To maximize fatigue life probably a little of each of the three approaches should be incorporated into the mix design.

Present Georgia DOT specifications require the asphalt concrete to be compacted to 96 percent of the theoretical maximum 50 blow Marshall Mix Design density. The easiest method to implement for improving the fatigue life would be to change the current specification to require in the field 98 to 100 percent of the 50 blow theoretical density. Preferably, however, the mix design procedure should be changed from 50 to 75 blows which would more closely simulate the condition of field compaction that would result from heavy traffic loadings. Also, the asphalt content should in general be increased a small amount above the presently used optimum. To minimize the possibility of future rutting problems, the asphalt content could at first be increased by a relatively small amount above the value that would presently be used. After constructing the pavement, periodic inspections could be made to evaluate the effect of the increased asphalt content on rutting. Of course, in designing the mix, the rutting potential should be investigated using the proposed test procedure if concern exists about future rutting of the mix. The fatigue test results have clearly shown that even an increase in asphalt content of 0.25 percent has the potential of increasing the fatigue life of the mix by 50 to 100 percent. Finally, when gradations are used which may not result in high fatigue resistance (due to high air void content), the gradation should be either changed or the fatigue life of the proposed mix should be evaluated using the test developed in this investigation. If the fatigue life is found not to be sufficiently great, the gradation, asphalt content and/or materials used in the mix should be changed.

Mixes requiring fatigue testing would include those having relatively large voids usually resulting from a coarse gradation, marginal materials, and some mixes having very fine gradations.

The test method and design criteria developed as a part of this investigation are for conventional mixes. In general the same approaches should be valid for marginal materials including sand asphalts and sand/stone blend asphalt concretes. However, caution should be exercised in using the procedures and design criteria presented in this investigation for the design of mixes utilizing this type of materials. In some instances, the design criteria and test methods for marginal materials may have to be modified to take into consideration the possible poorer performance of these mixes.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

A mix design procedure was developed for asphalt concrete base course materials. This procedure utilizes the conventional Marshall Mix Design Method and also includes tests for evaluating the rutting and fatigue characteristics of the mix. The proposed procedure places more emphasis on maximizing the fatigue life of the mix while at the same time limiting rutting to an acceptable level. In general the fatigue performance of a mix can be satisfactorily controlled by limiting the air voids in the mix and using a sufficiently high asphalt content. Therefore fatigue and rutting tests need not be performed for every mix design. Conditions where rut tests should be performed include when high asphalt contents or marginal materials are used. Fatigue tests should be performed on mixes using marginal materials and in some instances on mixes having air void contents greater than approximately 5 to 6 percent such as some mixes with coarse and fine gradations.

The proposed fatigue test consists of placing a 3 in. by 3 in. (76 by 76 mm) by 20 in. (508 mm) long beam specimen on an elastic subgrade having a modulus of subgrade reaction of 284 pci (7,860 gm/cc). A 110 lb. (490 N) repetitive load is cycled until fatigue failure occurs. The test is performed at a temperature of 80°F (27°C). A mix with acceptable fatigue characteristics should be able to withstand approximately 175,000 to 200,000 repetitions of loading. The rutting test consists of performing a repeated load triaxial test on a cylindrical specimen of base course mix. The specimen is subjected to 100,000 repetitions of load at a temperature of 95°F (35°C). A confining pressure is used of either 3 or 5 psi (21 or 34 kN/m^2). The potential permanent deformation in the base course mix can be estimated by multiplying the measured permanent strain at 100,000 repetitions by the thickness of asphalt concrete layer and the factor λ. The factor λ which considers that the test is performed for only 100,000 load repetitions and also serves as a correlation factor with field performance is approximately 4.5 for a confining pressure of 5.0 psi (34 kN/m^2) and deviator stress of 25 psi (179 kN/m^2), and close to 1.7 for a confining pressure of 3.0 psi (21 kN/m^2). For both the fatigue and rutting test, it is desirable to test two identical specimens and use the average test results.

Fatigue

A large number of fatigue and rutting tests were performed on a wide range of black base and modified B base course mixes. Specific conclusions concerning the fatigue characteristics of the mixes tested are summarized as follows:

1. The fatigue life of an asphalt concrete base course mix is very sensitive to a number of mix design variables. Void content and asphalt content, however, are the primary independent variables affecting fatigue. Therefore, the fatigue characteristics of the mix can generally be controlled by controlling the void and asphalt content of the mix. Secondary variables having the most influence on fatigue performance were found to be number of blows used to compact the specimen in the Marshall Mix Design Method, aggregate gradation, asphalt viscosity, and aggregate type.

2. Air voids in the mix for practical purposes are the most important single variable affecting fatigue performance. Fatigue life was found to be inversely proportional to air voids content on a log-log plot. Therefore, in the range of 3 to 8 percent air voids, a small increase in voids significantly decreases the fatigue life of the mix.

3. Any mix variable which influences the air voids content of the mix significantly affects the fatigue life.

4. A small increase in asphalt content of a mix can greatly increase the fatigue life. For example, an increase in asphalt content from 4.5 to 4.75 percent was found to approximately double the fatigue life of a granitic gneiss mix. Although the beneficial effect of a small increase in asphalt content became less with increasing asphalt content, the fatigue life was still significantly increased up to asphalt contents of at least 5.5 percent.

5. Using a 75 blow Marshall Mix Design Method rather than the presently used 50 blow design resulted in a significant increase in fatigue life between the range of asphalt contents investigated of 4.0 and 5.5 percent. The beneficial effect is due primarily to a decrease in void content and became less at a decreasing rate with an increase in asphalt content. Even at an asphalt content of 5.5 percent, however, the fatigue life of the 75 blow mix was found to be five times greater than the 50 blow mix.

6. Compaction of the asphalt concrete mix under traffic, hardening of the asphalt cement, and other environmental factors greatly complicate interpreting how a mix having lower initial air voids would actually perform in a pavement compared with mixes having higher air void contents. An analysis developed to consider the effects of traffic compaction indicated that going from the presently used 50 blow mix to 75 blows should result in an increase in fatigue life varying from approximately 25 to 70 percent depending on the actual fatigue life of the pavement.

7. The fatigue life of some heavily trafficked pavements in Georgia has been observed to be approximately 8 to 10 years. For pavements similar to these, going to a 75 blow Marshall Mix Design has the potential for increasing the fatigue life by 50 to 70 percent. Decreasing the initial air voids content of the mix by changing other variables such as gradation would have as much or even more beneficial effect on fatigue performance.

8. Use of an AC-40 asphalt cement decreased the fatigue life of a granitic gneiss mix (50 blow Marshall Mix Design) compared with a similar mix prepared with AC-20.

9. Substitution of flyash filler for granitic gneiss mineral filler (portion less than the No. 200 seive), resulted in a fatigue life for the mix investigated approximately 1/4 that of the granitic gneiss mix.

10. Portland cement mineral filler when used in a mix instead of granitic gneiss fines did not have a significant effect on the fatigue life of the mix investigated.

11. Black base mixes having fine and coarse gradations near the allowable specification limits had significantly lower fatigue lives at equal asphalt contents than the standard mix used in this investigation. At an asphalt content of 4.8 percent, the standard mix had 4.1 percent voids, the fine mix 5.9 percent voids and the coarse mix 8.3 percent voids. The difference in performance of

these mixes is primarily related to the difference in air voids.

12. The fatigue life of a limestone aggregate mix was less than 1/2 that of a similar mix using granitic gneiss aggregate. Once again the difference in air voids content accounts for most of the variation in fatigue life.

Rutting

Specific conclusions developed from the results of the repeated load triaxial test and Shell creep test results are summarized as follows:

1. The rutting tests indicate that moderate changes in mix variables do not tend to greatly affect the rutting characteristics of the mix. Indeed, the Shell creep test results indicated that increasing the asphalt content from 4.5 to 5.5 percent should not have any significant effect on the rut depth of the standard granitic gneiss mix. The repeated load triaxial test, however, indicated that going from 4.5 to 5.5 percent could increase the rut depth a modest amount of about 16 percent.

2. The repeated load and Shell creep test results indicated that the limestone aggregate mix tested should have approximately 18 and 35 percent, respectively, less rutting than a similar granitic gneiss mix.

3. The repeated load test results indicated that the mix prepared using AC-40 asphalt cement at asphalt contents in the vicinity of 5 percent should undergo approximately 20 percent less rutting than a similar mix prepared using AC-20.

4. The repeated load and Shell creep test results indicated that a black base mix prepared with a fine and coarse gradation should undergo approximately the same amount of rutting as the standard gradation mix. The Shell creep results did, however, indicate that the fine gradation mix might undergo slightly more rutting.

5. Use of portland cement and flyash mineral filler did not significantly change the rutting properties observed in the triaxial test.

6. The results of both the repeated load and Shell creep tests indicated that a 75 blow Marshall mix would have approximately the same rutting characteristics as a 50 blow mix.

7. The repeated load test results indicated that the stone-sand blend (I-95 mix) investigated might undergo as much as twice the amount of rutting experienced in the standard granitic gneiss mix. The I-95 mix failed when subjected to the standard compressive stress of 15 psf (100 kN/m^2) used in the creep tests. These specimens were the only ones to fail at this stress level during the creep testing program. Use of an 8 psi (55 kN/m^2) stress level and a specimen having a height to diameter ratio of approximately one was found not to result in specimen failure. The creep test results obtained using the lower stress level indicated that the I-95 material might rut as much as 2-1/2 times the standard granitic gneiss mix.

Recommendations

The following recommendations were developed from the investigation of the fatigue and rutting properties of asphalt concrete base course mixes:

1. The air voids in an asphalt concrete mix have a controlling effect on fatigue performance. To maximize fatigue life, the air voids in a base course mix should be as small as possible and limited where practical to a maximum of 6 percent and preferably 4 to 5 percent. To prevent bleeding, the air voids should not be less than approximately 2 percent.

2. The fatigue life of a mix can be very significantly increased by a small increase in asphalt content. The recommendation is therefore made that the asphalt content be increased above the presently used optimum by a relatively small amount. All that needs to be done is to slightly change the presently used procedure for selecting the Marshall Mix Design optimum. Even a moderate increase in asphalt content should not result in a significant increase in rut depth, but should greatly increase the fatigue life of the mix.

3. To optimize fatigue life, either the Marshall Mix Design Method should be changed from the presently used 50 blow procedure to 70 or 75 blows, or else a higher level of compaction should be specified in the field. Decreasing the initial void content of the mix has the potential for significantly increasing the fatigue life of base course mixes.

4. The recommended modified Marshall Mix Design Procedure should be adopted as a routine design method. This method should include changing the procedure for selecting the optimum asphalt content so as to obtain a slightly higher optimum. Also, where practical the mix should be designed to have an air voids content equal to or greater than 2 percent and less than preferably 5 percent. The air voids should be kept as low as practical but greater than the minimum value.

5. Figures 36, 38 and 39 given in Chapter 7 can be used to estimate the effect of asphalt content and air void content on fatigue life. For most mixes the effect of these primary variables which affect fatigue life (asphalt content and air voids content) can be determined using these figures. Therefore, in general fatigue and rutting tests need only be performed on mixes involving unusual materials, gradations or asphalt contents. The fatigue and rutting tests developed as a part of this investigation should be implemented for evaluating the physical properties of these special mixes. Asphalt concrete mixes where fatigue and rutting tests could be used include mixes having high air void contents, mixes with low asphalt contents (fatigue) and high asphalt contents (rutting), and marginal materials.

6. The fatigue performance of a mix is also closely related to the ring and ball softening point temperature. The asphalt cement used in a mix should have a minimum ring and ball softening point temperature of 118°F (48°C) to insure good fatigue performance.

7. During mixing the asphalt cement may be significantly hardened due to abnormally high temperatures or excessive mixing times. Higher viscosity asphalts caused by hardening during mixing will result in a significant reduction in fatigue life. If this is found to be a problem, specifications for controlling the viscosity or penetration after mixing should be developed to minimize the possibility of hardening during the mixing operation.

8. Both fatigue and rutting tests should probably be performed on mixes utilizing marginal materials at least until sufficient experience and adequate specifications are developed to insure satisfactory fatigue performance and an acceptable level of rutting. Some modifications of the test procedures including a slightly softer rubber pad in the fatigue test will probably be required.

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APPENDICES

APPENDIX A

ESTIMATION OF RUT DEPTH FROM CREEP TEST RESULTS BY THE SHELL METHOD

ESTIMATION OF RUT DEPTH

The following discussion summarizes the procedures used to calculate rut depth from the results of the Shell creep test. The procedure used to estimate the rut depth from the creep test results follows the method developed by the Shell-Laboratorium, Amsterdam [40-44].

Estimation of Rut Depth From Creep Test Results

The rut depth calculated for the 10.5 in. (267 mm) thick asphalt concrete layer from the results of the Shell creep test are for the following conditions:

a. An 18 kip (80 kN) single axle, dual wheel loading with the tires inflated to 85 psi (286 kN/m^2). The corresponding radius of the circular loaded area is 4.105 in. (104 mm).

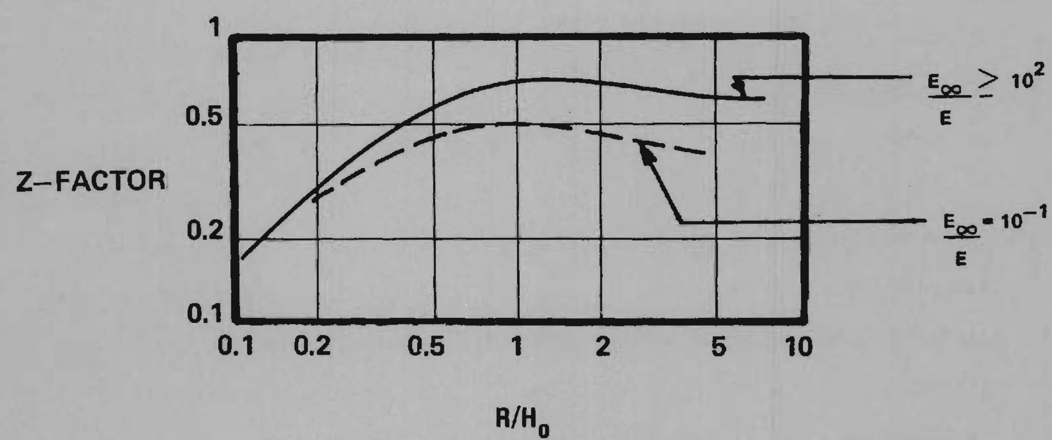
b. An equivalent number of 18 kip (80 kN) axle loads, N equal to 1×10^6 applied at an average asphaltic concrete pavement temperature of 95^oF (35^oC).

c. A vehicle speed of 50 mph (80 km/hr.)

d. A pavement structural section consisting of 10.5 in. (267 mm) of asphalt concrete resting on a resilient subgrade. The modulus of elasticity of the asphaltic concrete layer is assumed to be between approximately 96,000 and 550,000 psi (6.6×10^5 to 38×10^5 kN/m^2). The modulus of elasticity of the resilient subgrade is 2000 psi to 6000 psi (13,800 to 41,400 kN/m^2).

The following steps summarize the procedure used to calculate the rut depth from creep test data for the pavement and loading conditions given above. The method, however is quite general and can be used for other structural and loading conditions.

1. Determine the "Z" factor which relates the tire contact pressure to the corresponding average unconfined vertical stress in the asphalt concrete layer. For an 18 kip (80 kN), dual wheel loading at a tire pressure of 85 psi (286 kN/m^2), the radius of the loaded area is 4.105 in. (104 mm). From Tables 18 and 19 of reference (47) for spring and summer temperatures, the average value of Z is 0.41 which compares favorably with 0.45 obtained from Fig. A-1. A Z value of 0.45 was used in the reduction of the creep test data. The contact stress, σ_o between the tire and pavement is 85 psi (286 kN/m^2). The contact stress times Z gives the average stress in the pavement under the moving load (σ_{avg}).
2. The correlation factor C_m accounts for differences between the static (creep) loading applied in the laboratory test and the dynamic loading applied in the field.



R = RADIUS OF LOAD CONTACT AREA

H_o = ASPHALT LAYER THICKNESS

E = DYNAMIC MODULUS OF THE ASPHALT LAYER

E_∞ = DYNAMIC MODULUS OF THE SUBGRADE

FIGURE A–1. DIAGRAM FOR DETERMINING Z–FACTOR (AFTER REF. 40)

The factor C_m also accounts for other errors in the method. This factor has been found to vary between 1.0 and 2.0 for mixes having widely varying characteristics. A C_m factor of 1.8 was used in this study.

3. The stiffness of the mix, S_{mix} used to calculate the rut depth of the mix (equation A-2) is defined as the stiffness of the mix when the stiffness of the bitumen, S_{bit} is equal to the viscous (non plastic) component of the bitumen stiffness. The stiffness of the mix, S_{mix} is calculated as follows:

   a. Calculate the loading time, t, which is equal to the diameter of the loaded area divided by the vehicle speed:

$$t_o = \frac{2x4.104\ \text{in.}}{12\ \text{in./ft.}} \quad \frac{1}{73.4\ \text{ft./sec.}} = 0.00932\ \text{sec.}$$

   b. Calculate the Penetration Index (PI) from Fig. A-2. In this figure the temperature $T_{R\&B} - T$ is the difference between the ring and ball softening point temperature of the bitumen, $T_{R\&B}$ and the temperature at which the rut depth is being calculated. Both quantities are expressed in degrees of centigrade (^oC).

   c. Obtain the bitumen viscosity, η from the Van der Poel nomograph (Fig. A-3). The time of loading, t_o (step a) and penetration index, PI (step b) are used in this calculation.

   d. The viscous component of the bitumen stiffness, $S_{bit,visc}$ is then calculated from the following expression:

$$S_{bit,visc} = \frac{3\eta}{N\ t_o} \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad \text{(A-1)}$$

   where the loading time, t_o was calculated in Step 3(a), the bitumen viscosity, η in Step 3(c) and N is the equivalent number of 18 kip (80 kN) axle loadings applied at 95^oF ($N = 1 \times 10^6$ repetitions). The value of $S_{bit,visc}$ determined from equation (A-1) is set equal to S_{bit} and is used later in Step 3(f) to determine the appropriate stiffness of the mix to use in calculating the rut depth.

   e. Plot S_{bit} as a function of S_{mix} (at corresponding times) on log-log paper.

      (1) Values of the stiffness of the mix, S_{mix} are calculated at different times from the results of the creep test as follows:

$$S_{mix} = \sigma_1/E_p^c \quad . \ . \ . \ . \ . \ . \ . \ . \ . \ . \quad \text{(A-2)}$$

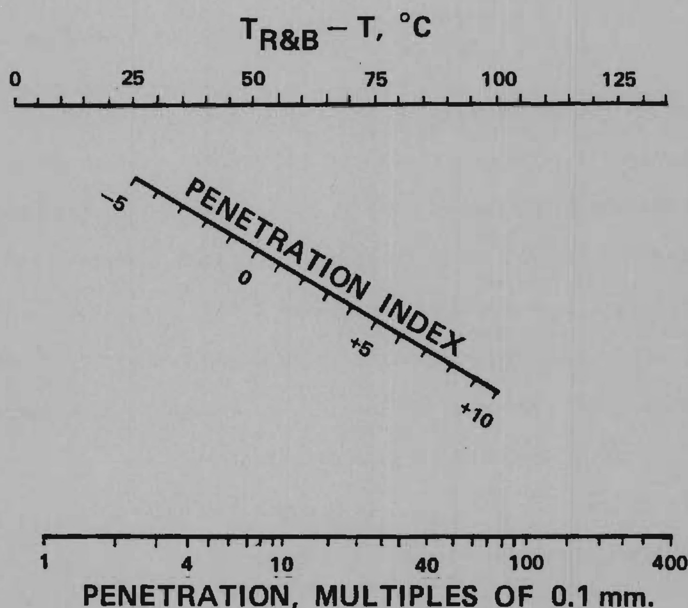


FIGURE A–2. DIAGRAM FOR DETERMING PENETRATION INDEX, PI (AFTER REF.40)

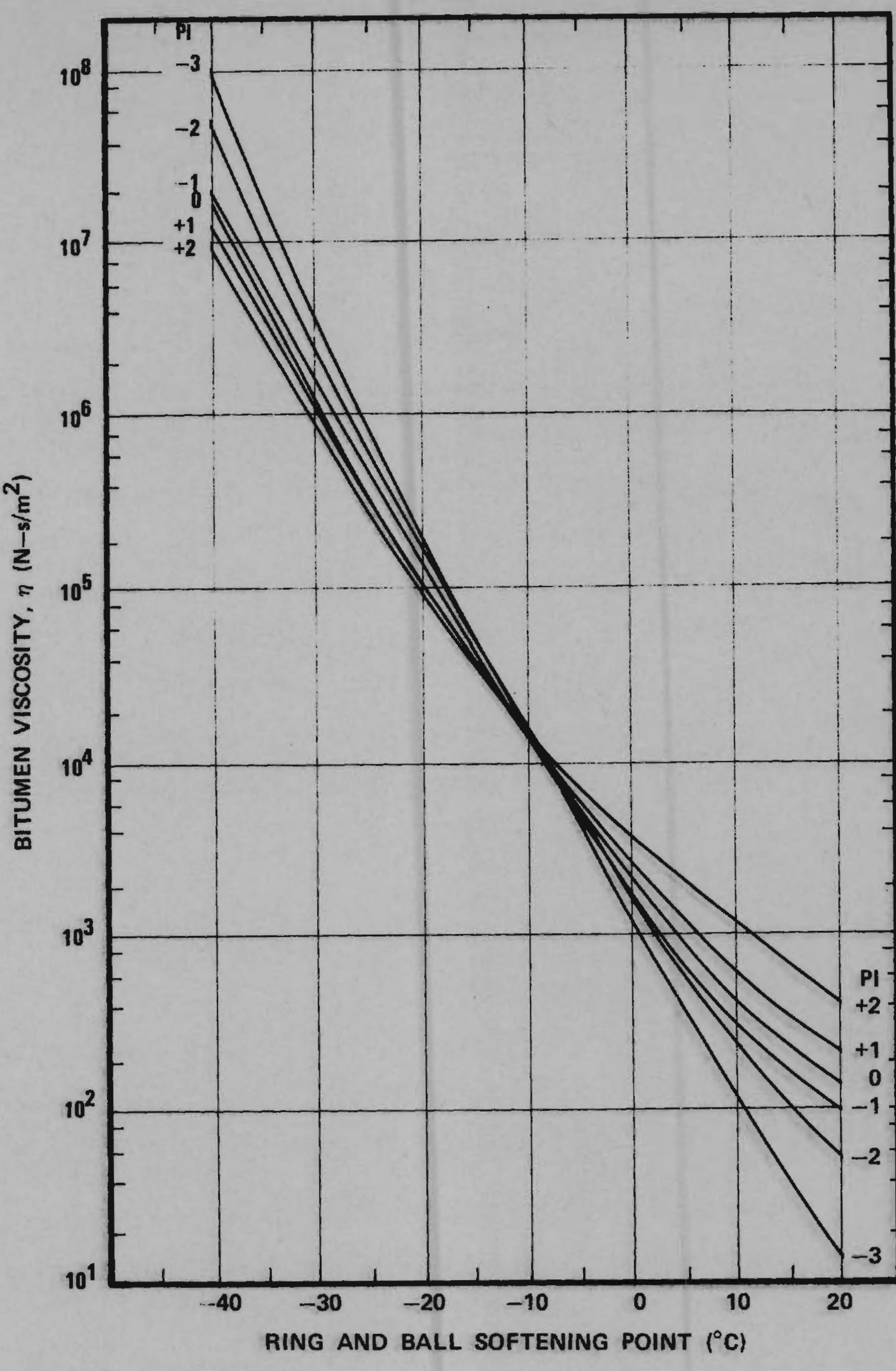


FIGURE A–3. DIAGRAM FOR DETERMINING BITUMEN VISCOSITY, η (AFTER REF. 40) where S_{mix} = stiffness of the mix

σ_1 = applied axial creep stress [15 psi (103 kN/m^2) for most of the tests].

E_p^c = total creep strain which has occurred in the specimen at the time under consideration. The total creep strain is equal to the recorder pen deflection times the calibration constant divided by the total specimen height.

(2) Values of S_{bit} are obtained at various arbitrarily selected times (knowing $T_{R\&B} - T$ and the PI for the given asphalt type) from the Van der Poel nomograph (Fig. A-4). The S_{bit} and corresponding S_{mix} values are then plotted on log-log paper and a straight line approximation is drawn through the points. The stiffness of the mix, S_{mix} to be used for the estimation of rut depth is obtained from this graph at an S_{bit} corresponding to the $S_{bit,visc} = S_{bit}$ that was calculated in Step (d).

4. All quantities required to calculate the rutting in the asphaltic concrete layer have now been evaluated. The rut depth is then calculated from

$$\Delta H = C_m H_o \left(\frac{\sigma_o Z}{S_{mix}}\right) \quad \text{(A-3)}$$

where ΔH = the permanent deformation (rutting) in the layer

H_o = initial layer thickness

The quantities Z and σ_o are determined in Step 1, C_m in Step 2, and S_{mix} in Step 3.

5. <u>An Example Illustrating the Calculation of Rut Depth From Laboratory Creep Test Data.</u>

I. Laboratory Creep Test Data [Step 3e]: The creep test was performed using an axial stress, σ_o = 15.03 psi on a specimen having a height, $\bar{H}$ = 8.03 in.

Time (Seconds)	Creep Deflection*δ ($\times 10^{-2}$ in.)	Permanent Strain ε_m (in./in.) $\delta/\bar{H}$($\times 10^{-3}$ in./in.)	S_{mix} σ_o/ε_m($\times 10^3$ psi)
100	1.86	2.31	6.49
1000	2.35	2.92	5.14
10,000	2.64	3.39	4.57

* Recorder pen deflection times the calibration constant.

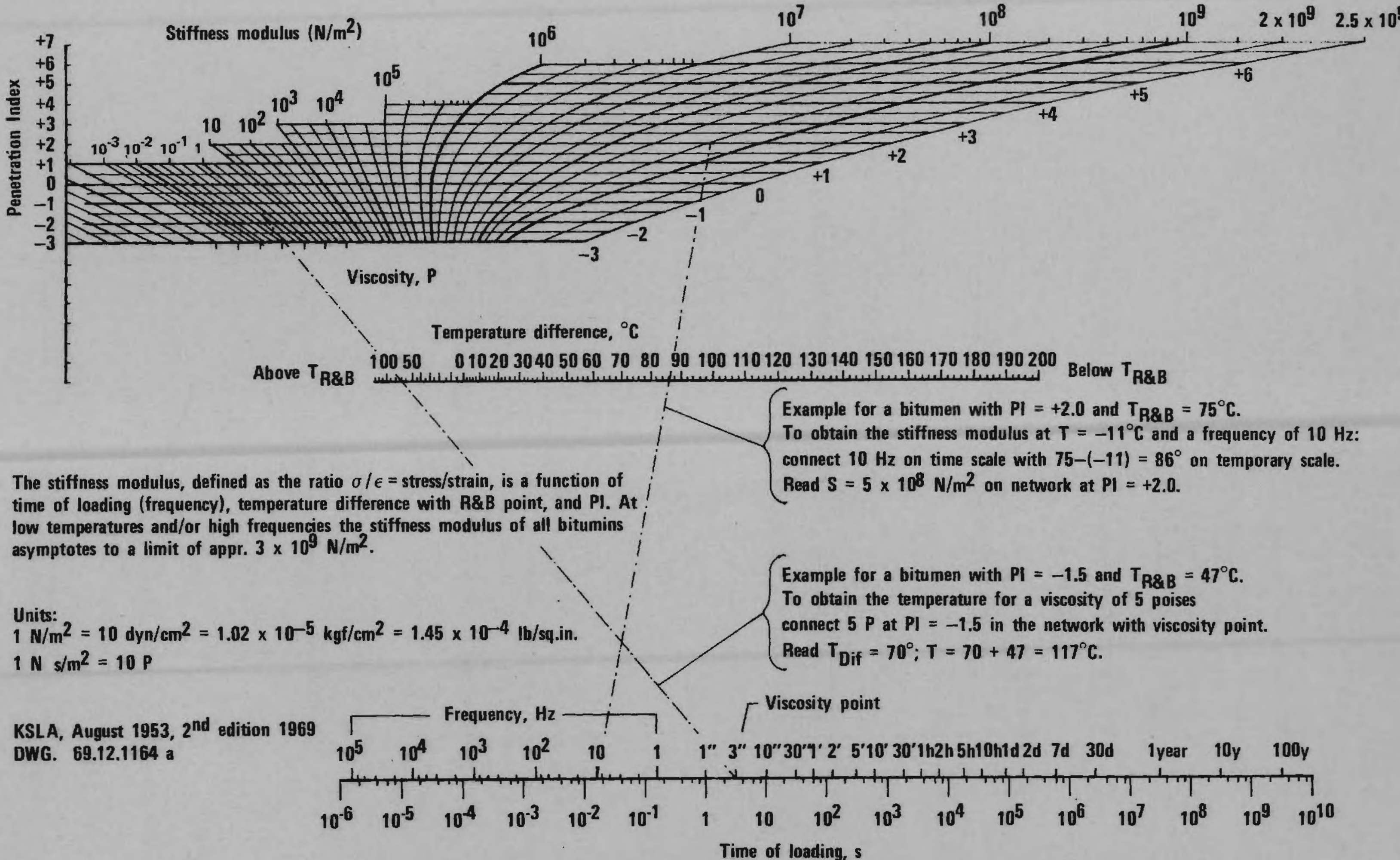


FIGURE A–4. NOMOGRAPH FOR DETERMINING BITUMIN STIFFNESS (AFTER VAN DER POEL)

II. Calculate the Penetration Index, PI and determine the bitumen viscosity given:

Creep Test Temperature T = 95^oF (35^oC)

Bitumen Properties of Material Tested:

Ring & Ball Softening Point, $T_{R\&B} = 124^oF$ (51.1^oC)

Penetration = 68

a. Determine the Penetration Index, PI from Fig. A-2 [Step 3b]

$$PI = -3$$

b. Determine the bitumen viscosity from Fig. A-3 [Step 3c]:

$$\eta = 8.1 \times 10^4 \text{ N-sec/m}^2 \text{ or } 11.75 \text{ psi-sec.}$$

c. From equation A-1 [Step 3d]:

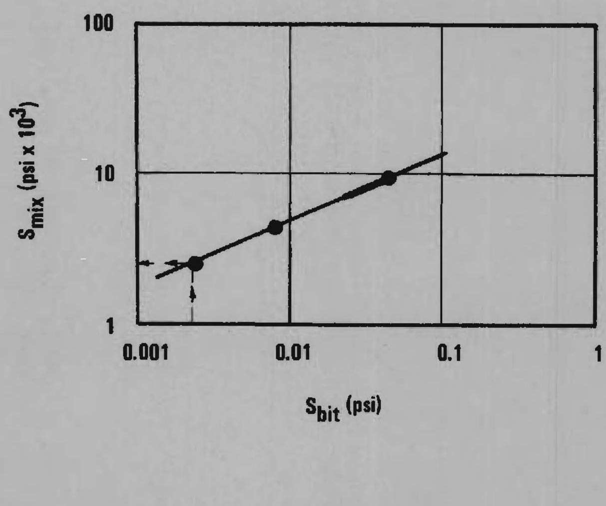
$$S_{bit,visc} = \frac{3\eta}{N\,t_o} = \frac{3(11.75 \text{ psi-sec})}{(1\text{x}10^6 \text{ rep.})(0.00932 \text{ sec.})} = 0.0038 \text{ psi}$$

Where as previously defined N is the equivalent number of wheel load repetitions and t_o is the wheel loading time (Step 3a).

III. Determine the stiffness of the bitumen, S_{bit} from Fig. A-4 [Step 3e(2)].

Time (sec.)	S_{bit} (psi)
100	~0.48
1,000	~0.048
10,000	~0.0048

IV. Plot S_{bit} (See Step III) as a function of S_{mix} (See Step I) for corresponding times on a log-log graph, and approximate the data with a straight line. Pick an S_{mix} corresponding to the calculated S_{bit} of 0.0038 psi (See Step IIc) [Step 3e].



V. Calculate from equation A-3 the reduction in the pavement layer thickness (rut depth) [Step 4]

$$\Delta H = C_m H_o \frac{Z \cdot \sigma_o}{S_{mix}} = \frac{1.8(85 \text{ psi})(10.5 \text{ in.})}{4.32 \times 10^3 \text{ psi}} = 0.167 \text{ in.}$$

APPENDIX B

REDUCTION OF FATIGUE TEST DATA

REDUCTION OF FATIGUE TEST DATA

The reduction of the fatigue test results consist of determining (1) the relationship between load and repetitions to failure, (2) the relationship between the tensile strain at 1,000 load repetitions and failure and (3) the estimated fatigue life of a given mix when used in an actual structural pavement section. The bending stiffness of the mix has an important effect on the resulting tensile strains that develops in a pavement structure when subjected to a repeated loading. *Since the bending stiffness varies for different mixes, a direct comparison of the fatigue curves is not necessarily indicative of the relative fatigue performance of the mixes when used in a pavement structure.* The relationship between the measured tensile strain in the beam specimen at 1,000 load repetitions and the number of repetitions to cause failure is presented in this report as a log-log plot. Because of the effect of varying mix stiffnesses, comparisons of the relative performance of different mixes should be made as discussed in the report using either the (1) relationship between load and repetitions to failure in the fatigue test or else (2) the estimated fatigue life of the mix in a selected pavement structure. Probably the former approach is most suitable for comparing fatigue performance of mixes.

1. Determine the Fatigue Curve: The fatigue curves give the relationship between the measured tensile strain in the beam specimen and the number of load repetitions to cause failure. The fatigue curves are usually presented as a log-log plot since the resulting relationship has been found to be approximately linear.

   a. Calculate the maximum tensile strain in the beam specimen at 1,000 repetitions. The tensile strain measured using a strain gauge is not the maximum since the gauge is located a finite distance from the bottom of the beam. The strain at 1,000 load repetitions is used since the strain measured at lower load repetitions has not stabilized to a relatively constant value. The strain is calculated as follows:

$$\varepsilon_t = C_c \cdot \delta_r^{pen} \left(\frac{H}{2L} \right)$$

   where

   ε_t = maximum tensile strain in asphaltic concrete beam specimen

   C_c = recorder calibration constant

   δ_r^{pen} = resilient deflection of the pen

H = thickness of the beam

L = distance from the neutral axis of the beam and the center of the strain gauge

b. Perform at least three or four fatigue tests on different specimens of the same mix. The tests should be performed at load levels selected to result in failure at approximately 5,000 to 10,000, 40,000 to 200,000 load repetitions. A wide variation in failure loads is necessary to satisfactorily define the slope of the fatigue curve.

c. Make log-log plots of both the load and strain as a function of the number of load repetitions.

2. Bending Modulus of Elasticity of Beam: The bending modulus of the asphalt concrete beam can be evaluated from both the measured deflection of the beam and the measured strain. Since the measured tensile strain is more closely related to bending than the vertical deflection of the beam, the measured tensile strain was used to calculate the bending modulus of the beam. The bending modulus of the beam, E_B can be computed as follows from the results of the fatigue test:

$$E_B = C'_{\lambda\ell} \left[\frac{(P\,\rho)^{4/3}}{4\,I\,k^{1/3}} \right] \quad . . . . \quad \text{(B-1)}$$

where E_B = bending modulus of the beam at the desired number of load repetitions

P = load applied to the beam

ρ = radius of curvature of the beam which is equal to the depth of the beam divided by two times the measured tensile strain

I = moment of inertia of the beam. For a rectangular shaped beam $I = BD^3/12$ where B is the beam width and D is the depth of the beam

k = modulus of subgrade reaction of the rubber pad measured by means of a plate load test. For the rubber pads used in these tests k was found to be 284 pci (7,861 gm/cc).

$C'_{\lambda\ell}$ = correction factor obtained from Fig. B-1

ℓ = length of the beam

Equation (B-1) is based on the beam on elastic foundation theory (Winkler theory). This theory assumes that the beam is elastic and is supported on a foundation consisting of a very large number of closely spaced, independent linearly elastic springs. The derivation of equation (B-1) is as follows:

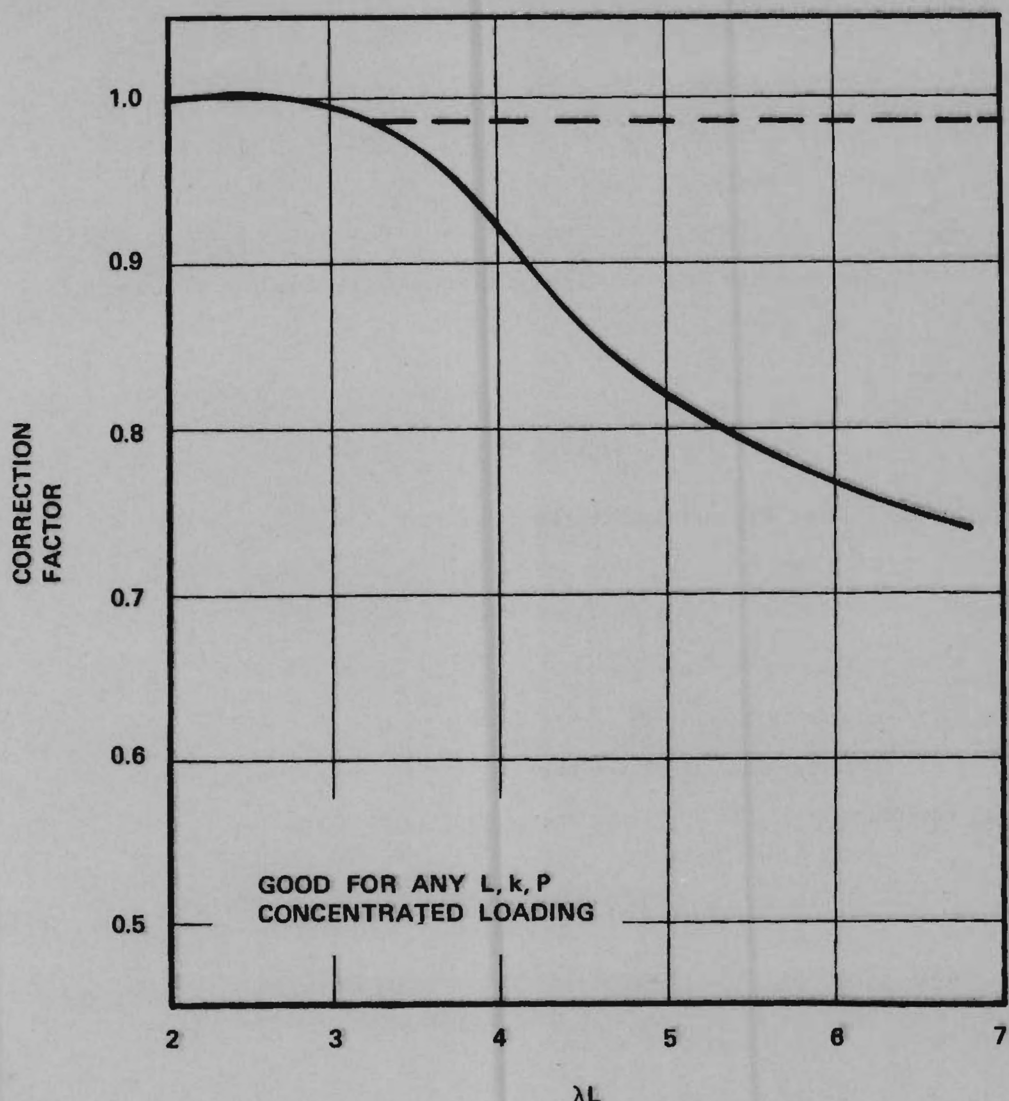


FIGURE B-1. CHART FOR DETERMINING CORRECTION FACTOR $C_{\lambda l}$ FOR CALCULATING BENDING MODULUS

Assume an infinite beam supported on a linear spring foundation and loaded by a concentrated load at the center

$$M = \frac{P}{4\lambda} C_1 \quad \text{. . . . .} \quad \text{(B-2)}$$

where M = moment at center of the beam

λ = $(k/4EI)^{1/4}$ where k is the subgrade modulus, k_o multiplied by the width of the B

C_1 = correction factor to account for the load actually being applied over a finite length of the beam and also to consider the length of the beam being finite

The maximum moment is then equal to

$$M = P/[4k/4EI)^{1/4}] \quad \text{. . . . .} \quad \text{(B-3)}$$

Assuming small deflections and linear material response

$$\frac{M}{E_b I} = \frac{1}{\rho} \quad \text{. . . . .} \quad \text{(B-4)}$$

where ρ = radius of curvature

E_b = modulus of elasticity of the beam

I = moment of inertia of the beam

Substituting equation (B-4) into (B-3) and rearranging terms gives

$$E = \frac{1}{4.I} \left[\frac{C_1^{4/3} \, (P\,\rho)^{4/3}}{k^{1/3}} \right] \quad \text{. . . . .} \quad \text{(B-5)}$$

The factor $C_1^{4/3}$ can be obtained from the available solution[(1)] for a beam of finite length subjected to a distributed loading in the center:

$$M = \frac{q}{\lambda^2} \left\{ \frac{\text{Sin}\lambda c \ \text{Sinh}\frac{\lambda \ell}{2} \ \text{Sinh}\lambda a \ + \ \text{Sinh}\lambda c \ \text{Sin}\frac{\lambda \ell}{2} \ \text{Sin}\lambda a}{\text{Sinh}\lambda \ell + \text{Sin}\lambda \ell} \right\} \quad \text{. . . .} \quad \text{(B-6)}$$

where q = symmetrical distributed loading

a = distance from the end of the beam to the edge of loading

2c = length of distributed loading

1. Hetenyi, M., BEAMS ON ELASTIC FOUNDATION, The University of Michigan Press, Ann Arbor, 1971.

The beam theoretically lifts off the subgrade support for $\lambda\ell \geq \pi$. Assume for this condition that the maximum effective beam length is $\lambda\ell = 3.14$. Considering the effect of the distributed loading and effective beam length, the correction factor $C_{\lambda\ell} = C_1^{4/3}$ can be developed using equation (B-6) and the expression for the maximum moment in a beam of finite length subjected to a single concentrated loading. For the beam length, (ℓ = 20 in.) and the width of loading (2c = 1.25 in.) used in the investigation, the correction factor $C_{\lambda\ell}$ is given in Fig. B-1 for $2 \leq \lambda\ell \leq 6$.

The bending modulus of elasticity of the asphaltic concrete beam can therefore be calculated from the expression

$$E_b = \frac{C_{\lambda\ell}}{4I}\left[\frac{(P\,\rho)^{4/3}}{k^{1/3}}\right] \qquad \text{(B-7)}$$

For a beam having a length of 20 in. and a 1.25 in. wide concentrated load in the center (the beam geometry used in this investigation) the appropriate values of $C_{\lambda\ell}$ to use in equation (B-7) are given in Fig. B-1. The radius of curvature, ρ is equal to $H/2\varepsilon_t$ where H is the depth of the beam and ε_t is the maximum tensile strain in the beam.

APPENDIX C

FATIGUE TEST RESULTS

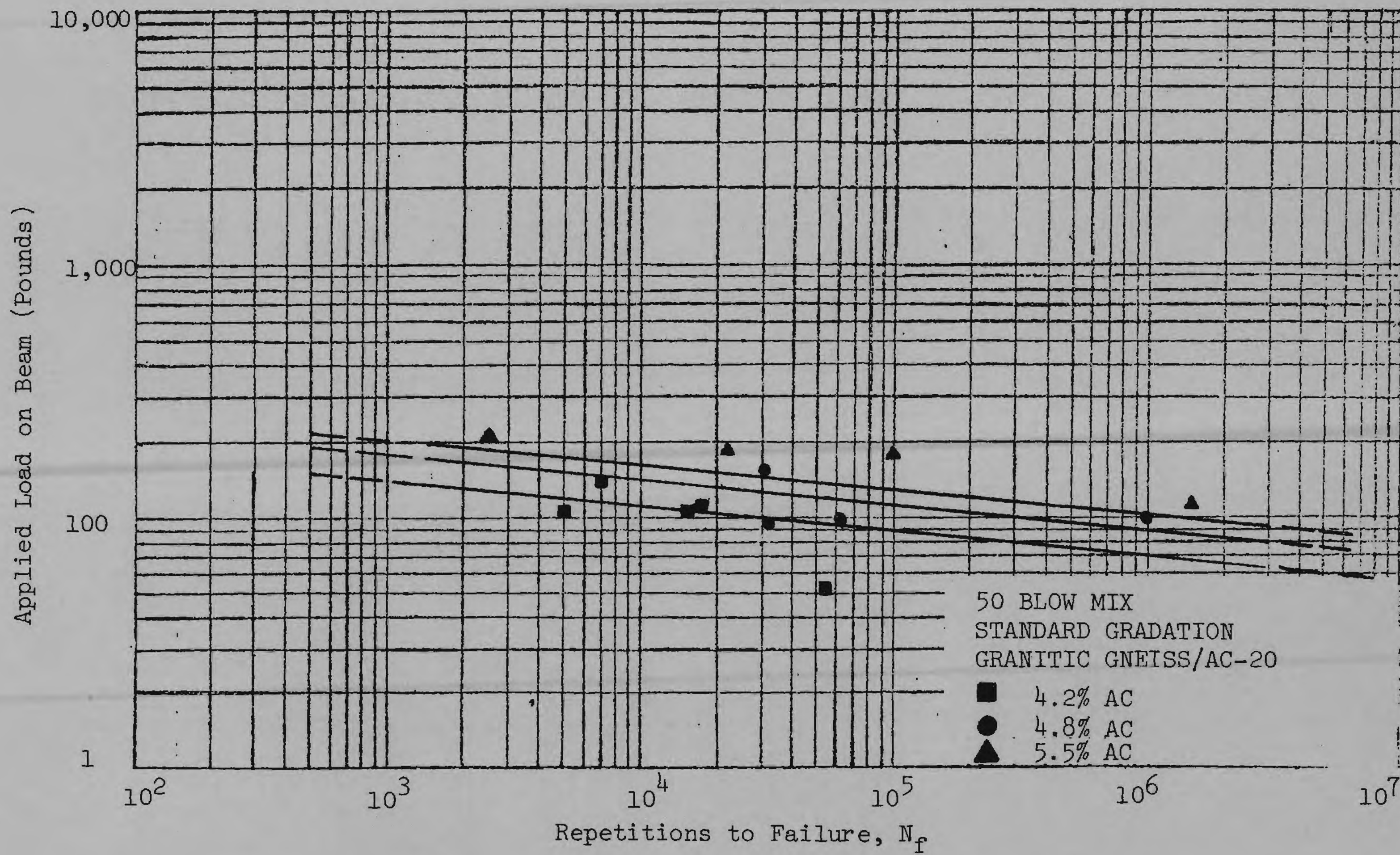


FIGURE C-1. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW BLACK BASE MIX.

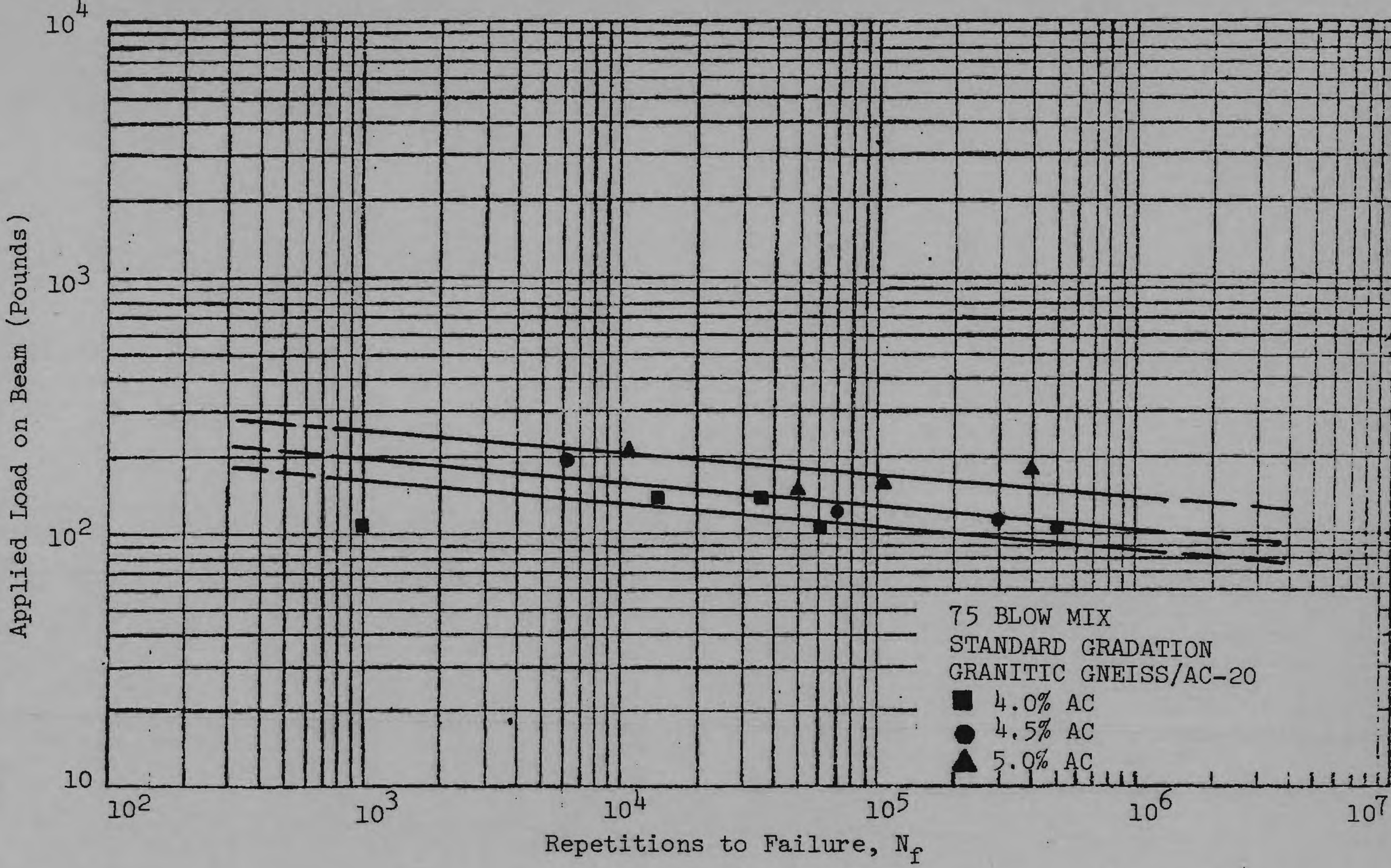


FIGURE C-2. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 75 BLOW BLACK BASE MIX.

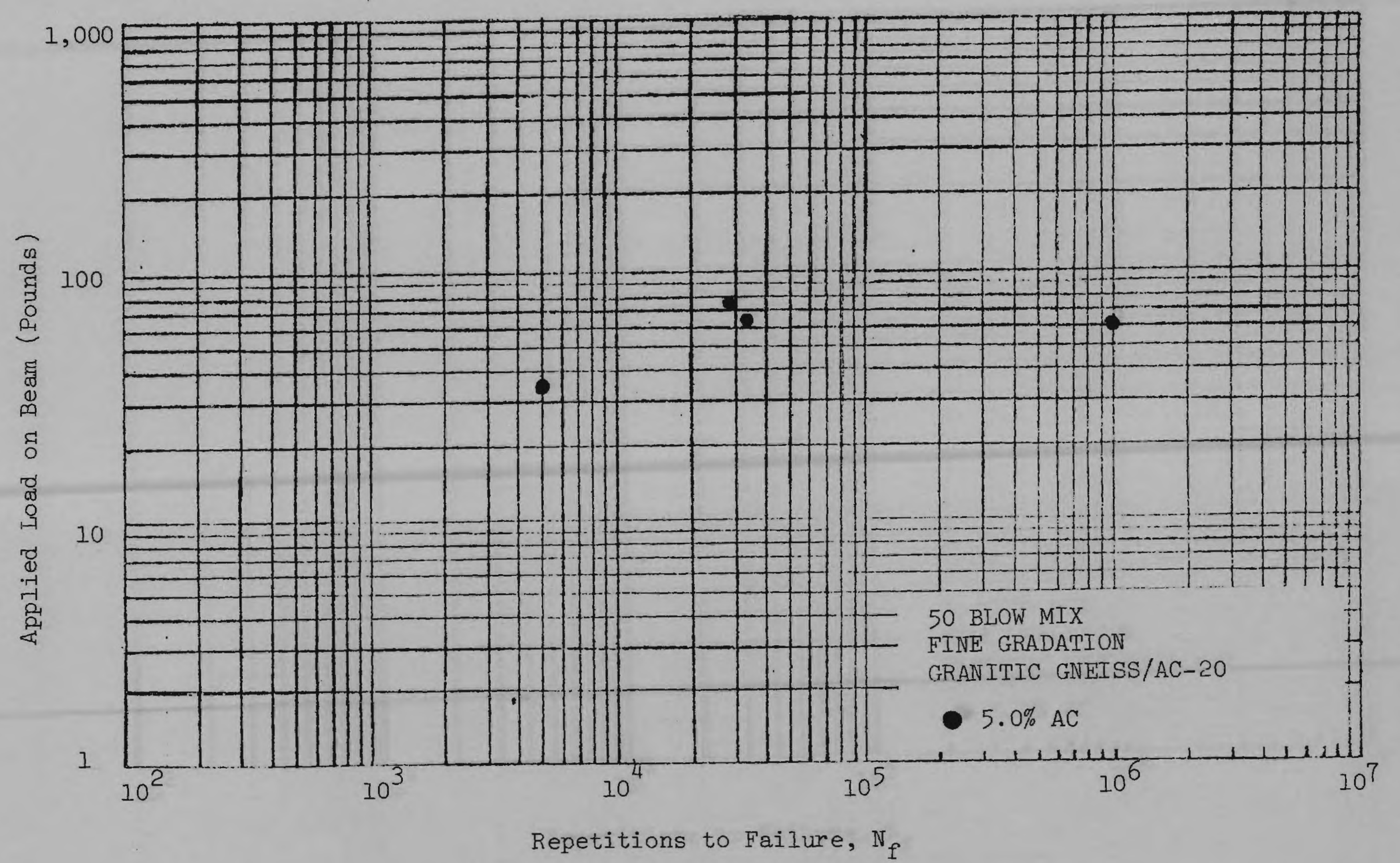


FIGURE C-3. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW BLACK BASE MIX WITH FINE GRADATION.

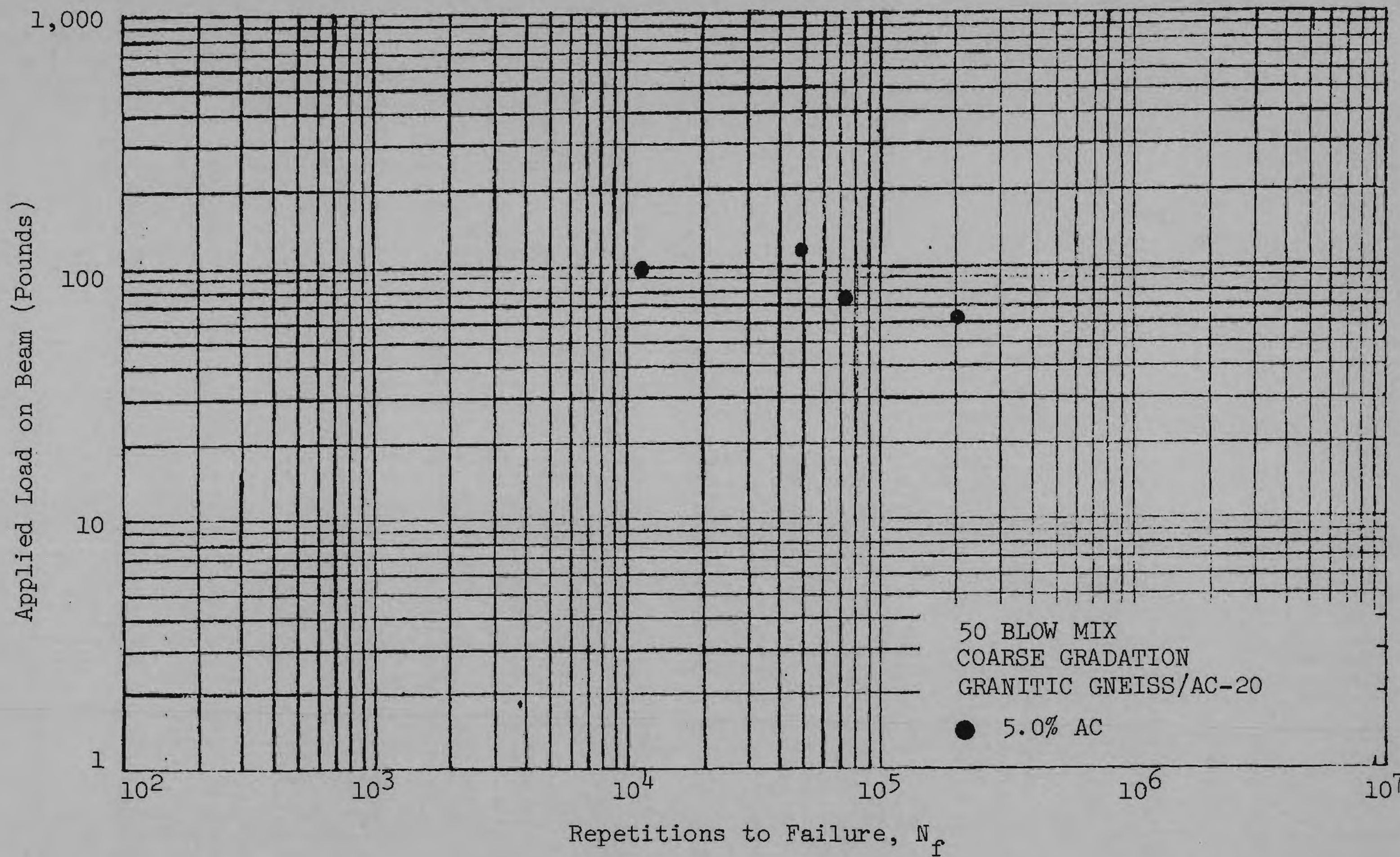


FIGURE C-4. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW BLACK BASE MIX WITH COARSE GRADATION.

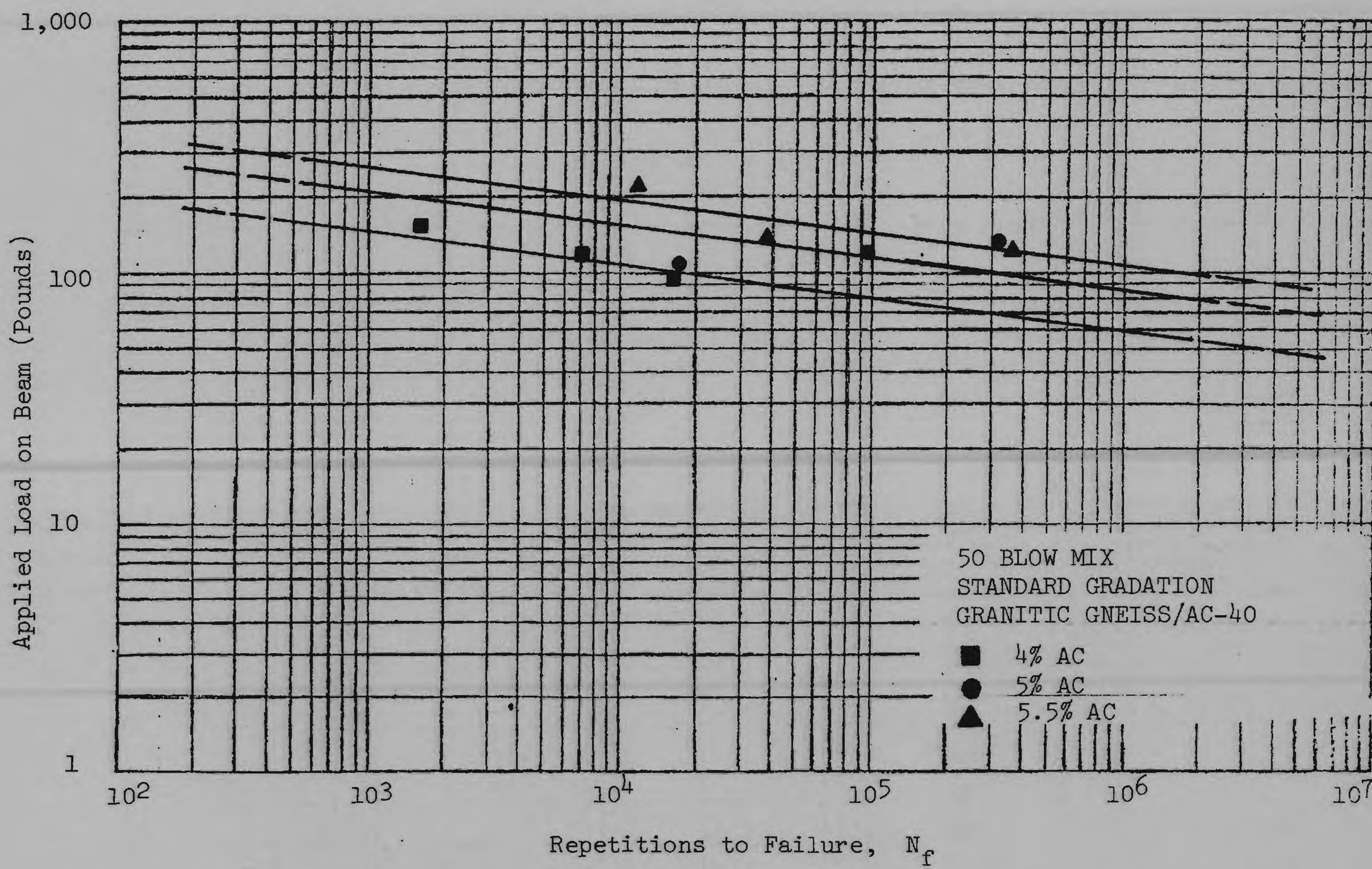


FIGURE C-5. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW BLACK BASE MIX WITH AC-40.

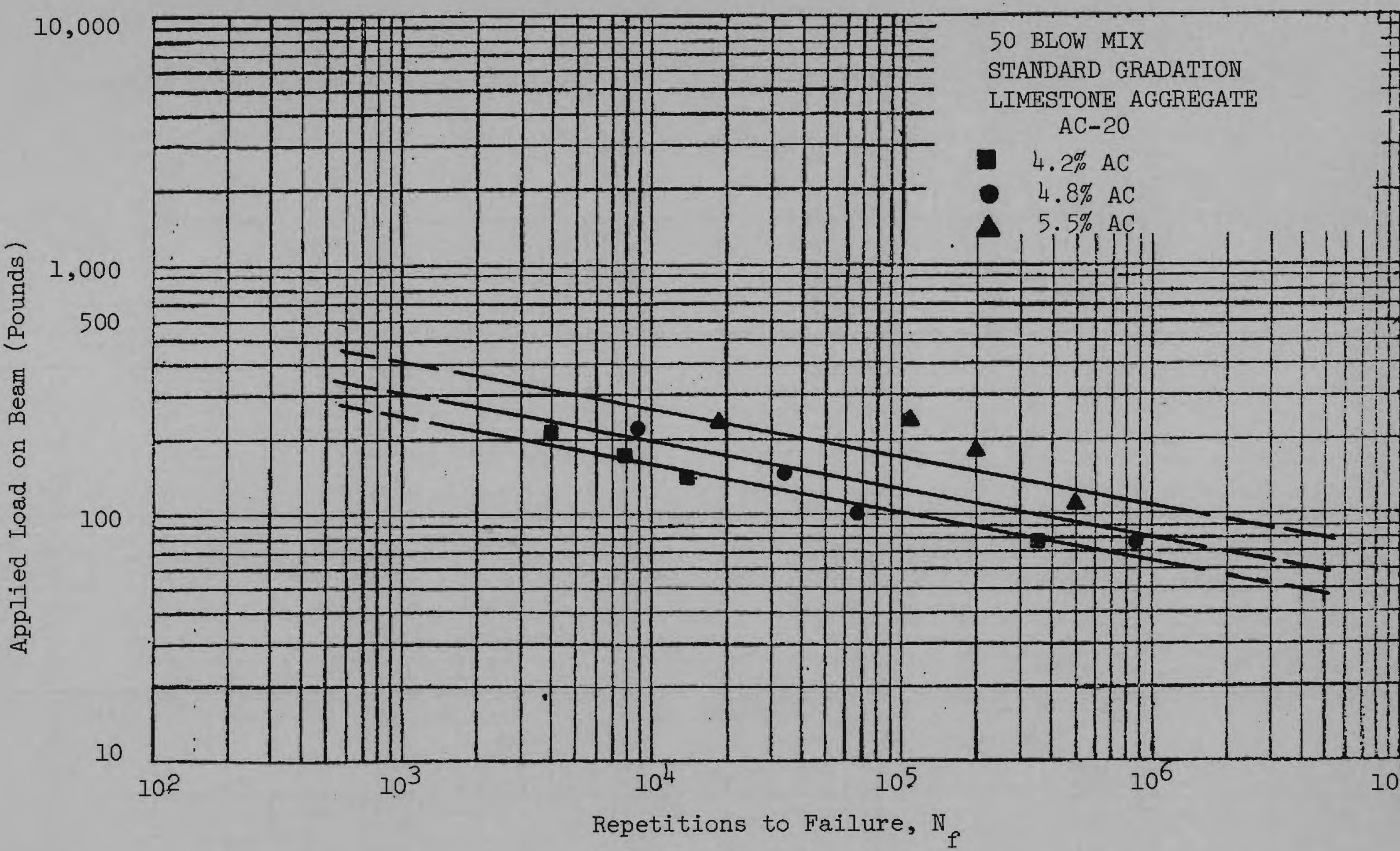


FIGURE C-6. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW BLACK BASE MIX WITH LIMESTONE AGGREGATE.

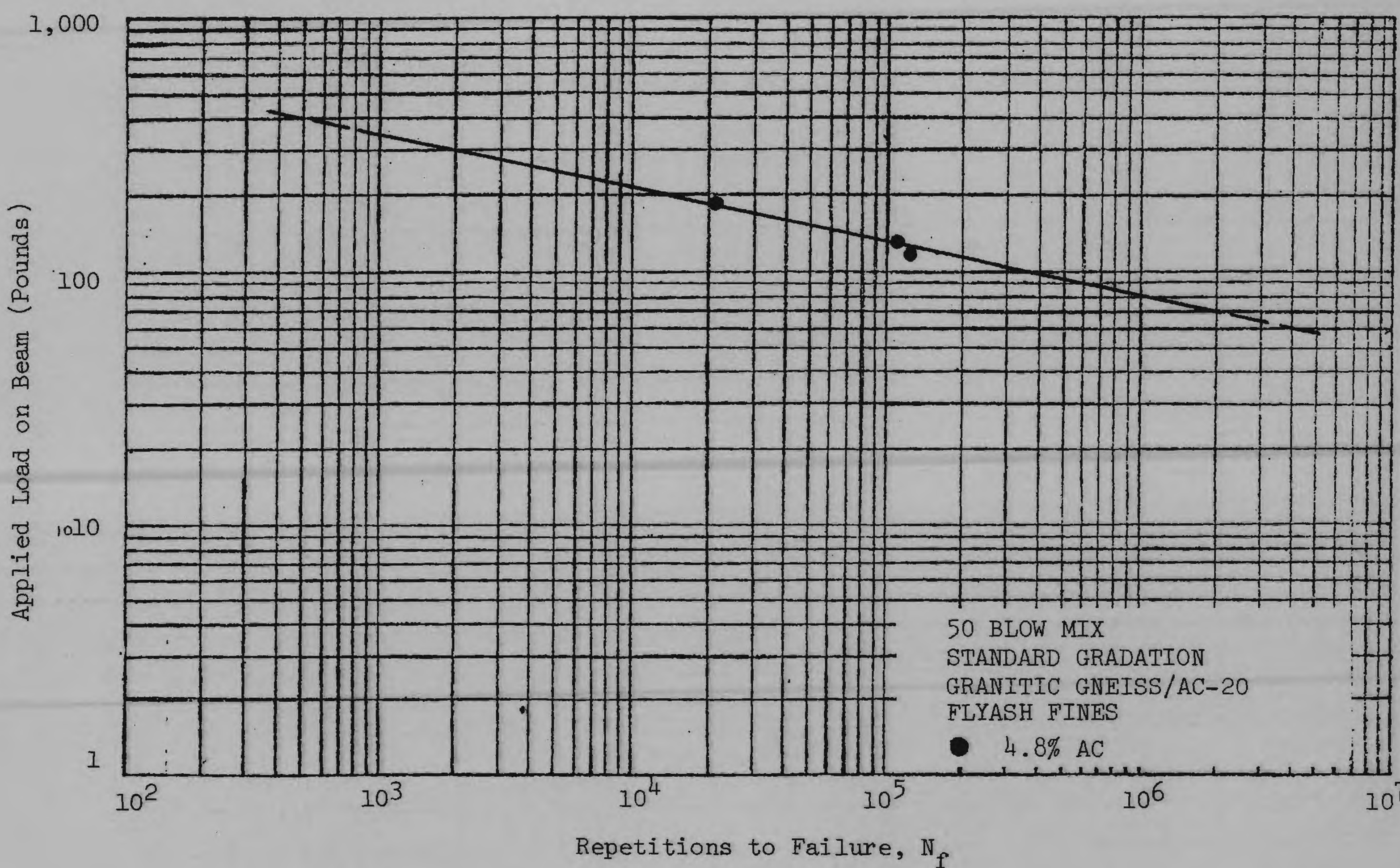


FIGURE C-7. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW BLACK BASE WITH FLYASH MINERAL FILLER.

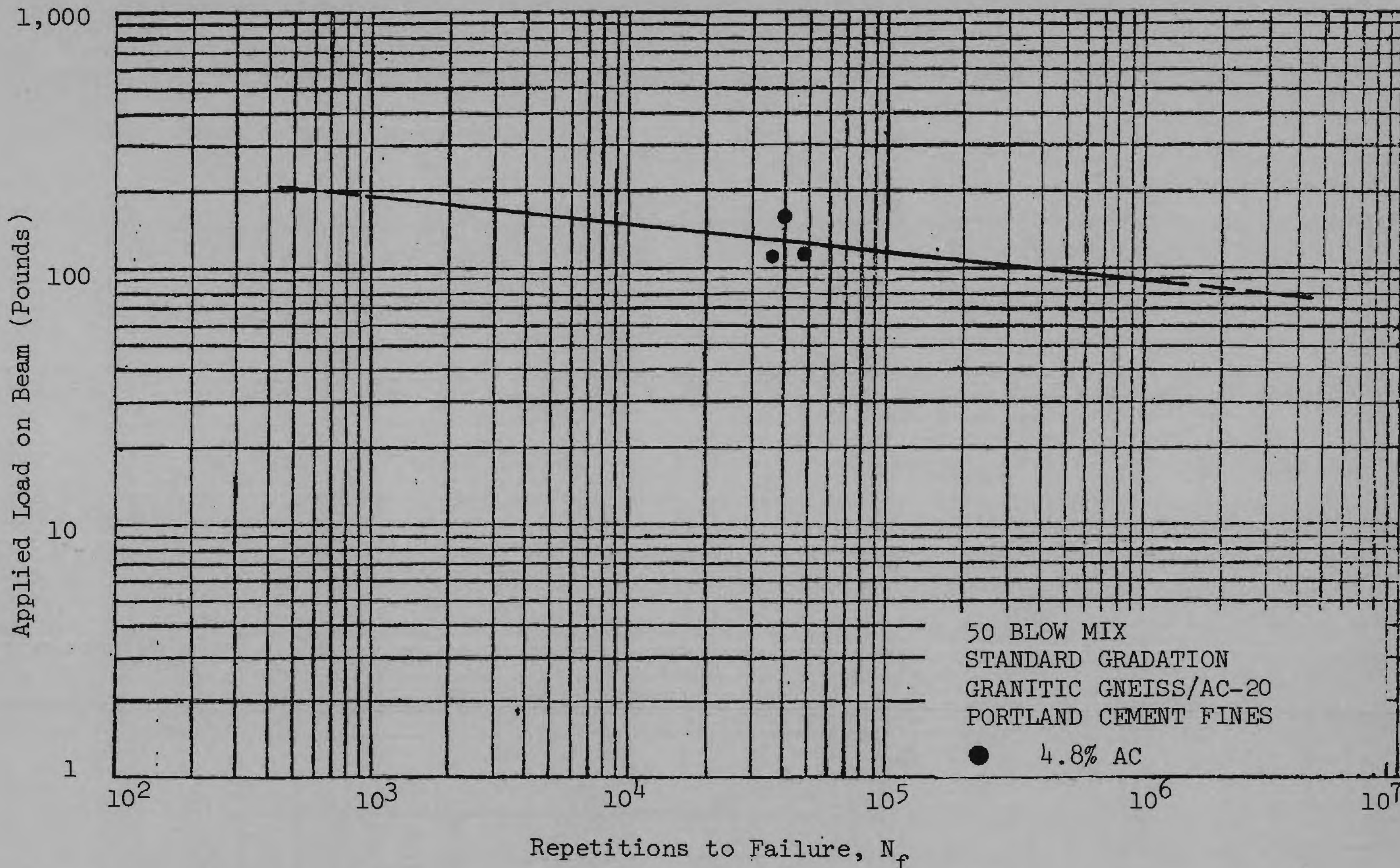


FIGURE C-8. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW BLACK BASE MIX WITH PORTLAND CEMENT MINERAL FILLER.

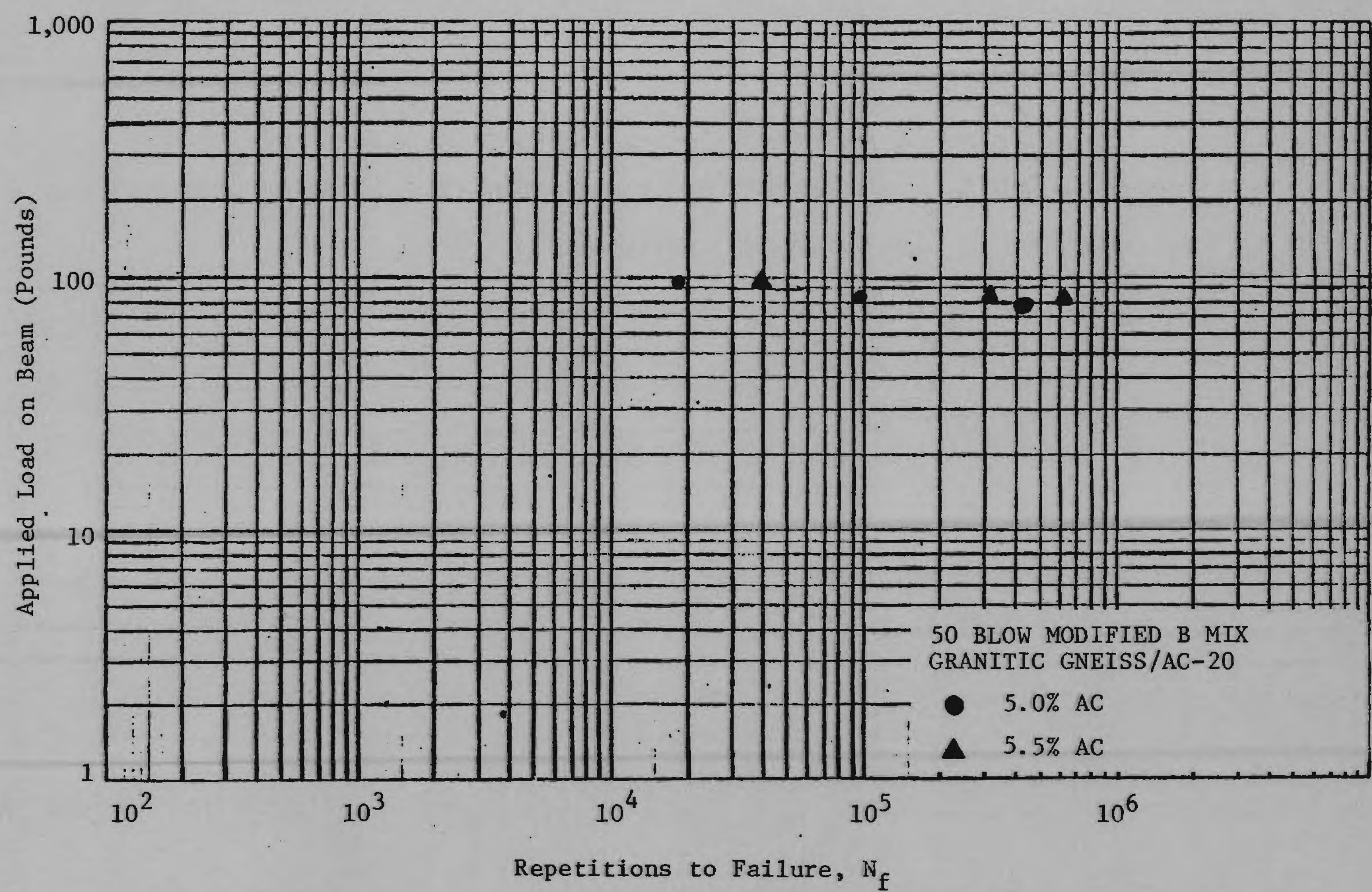


FIGURE C-9. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 50 BLOW MODIFIED B MARSHALL MIX.

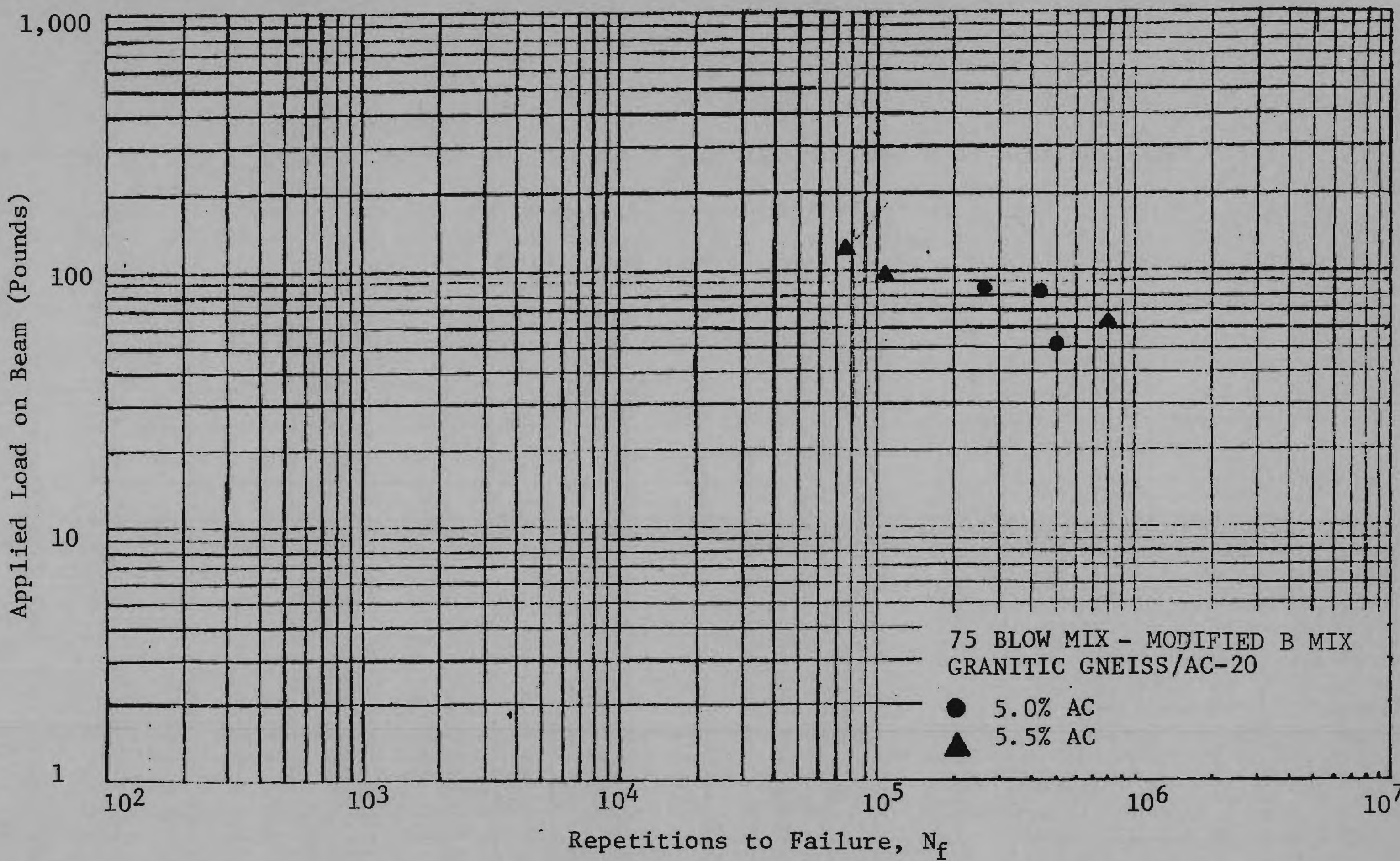


FIGURE C-10. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - 75 BLOW MODIFIED B MIX.

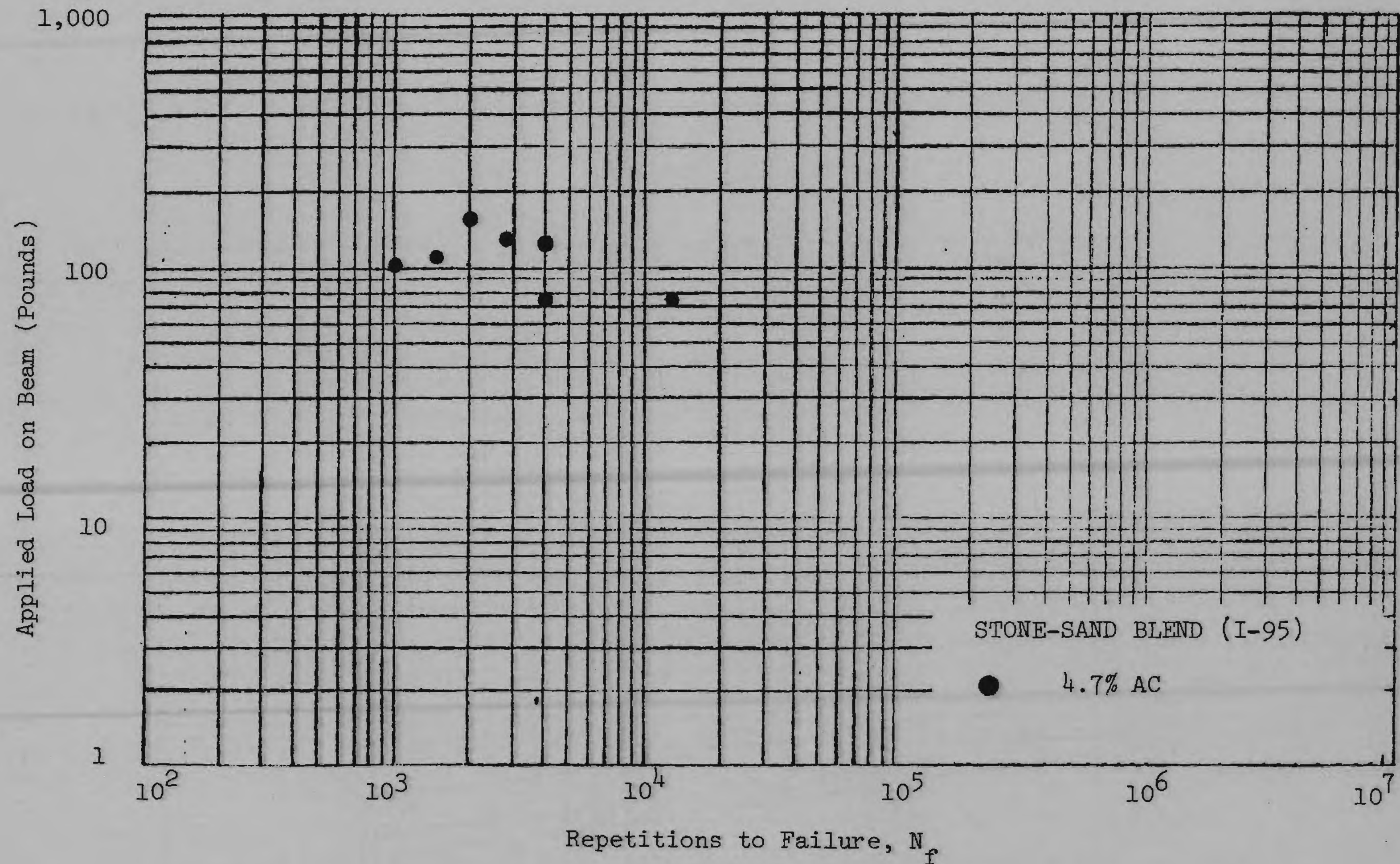


FIGURE C-11. REPETITIONS TO FAILURE AS A FUNCTION OF LOAD - STONE-SAND MIX (I-95).

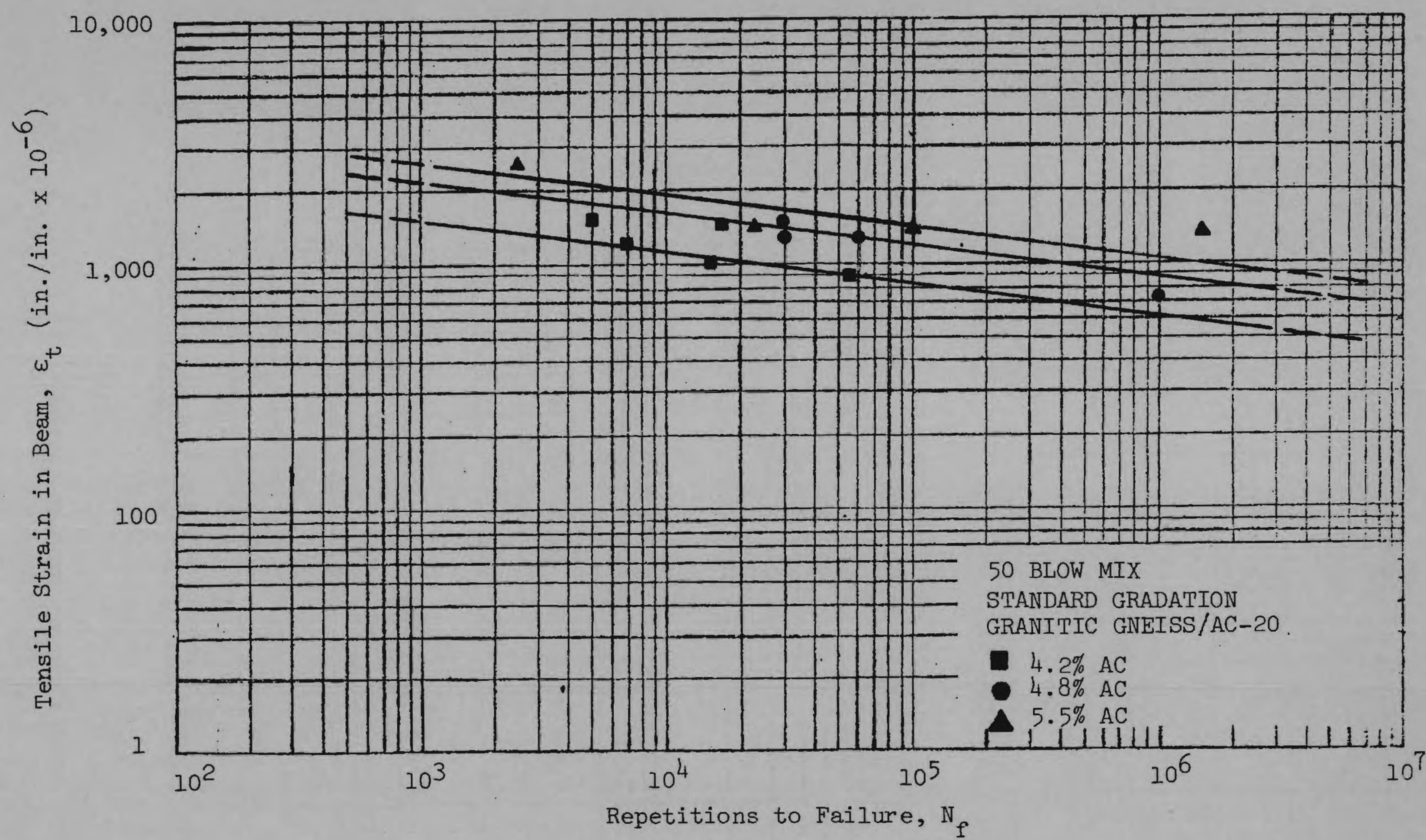


FIGURE C-12. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW BLACK BASE MIX.

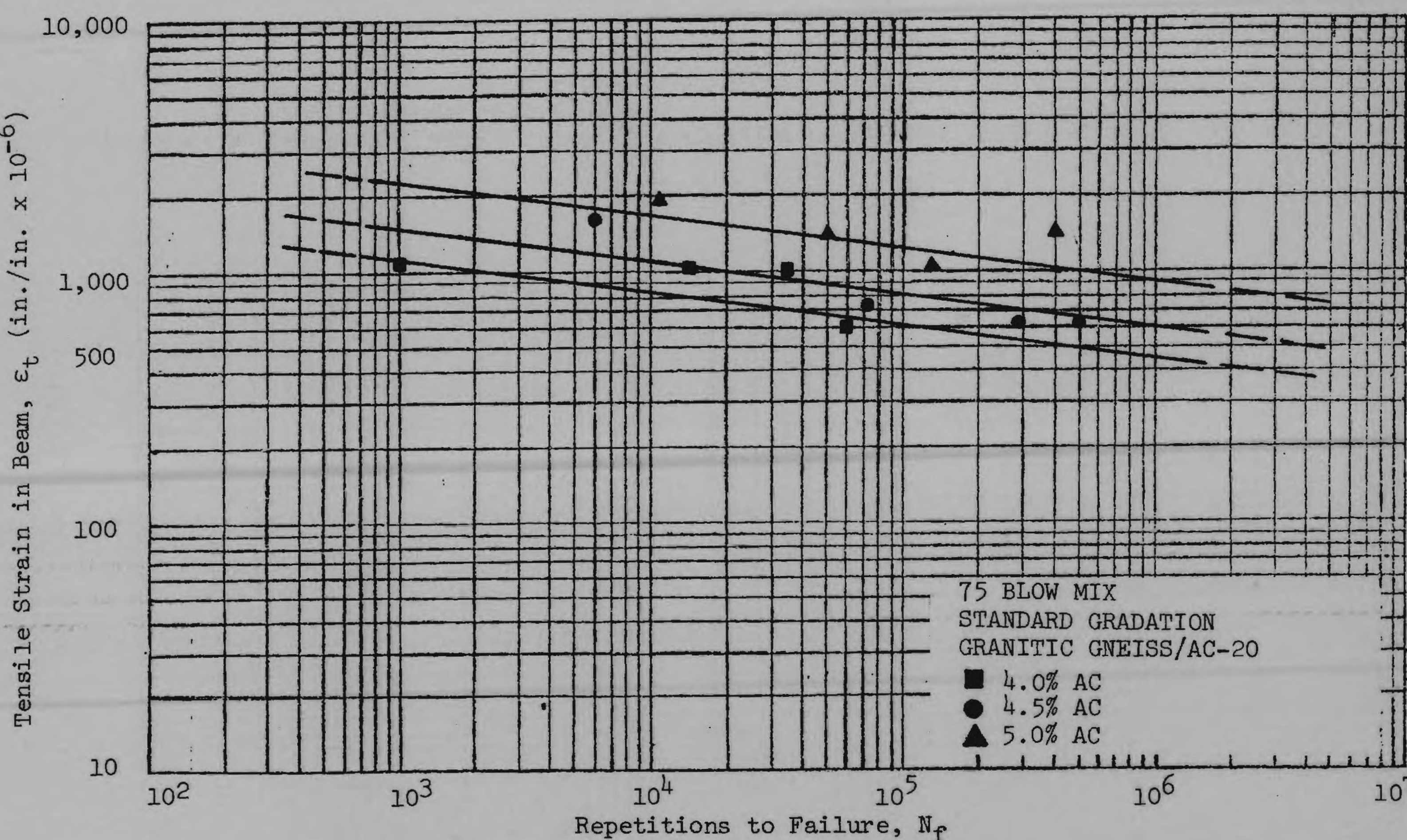


FIGURE C-13. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 75 BLOW BLACK BASE MIX.

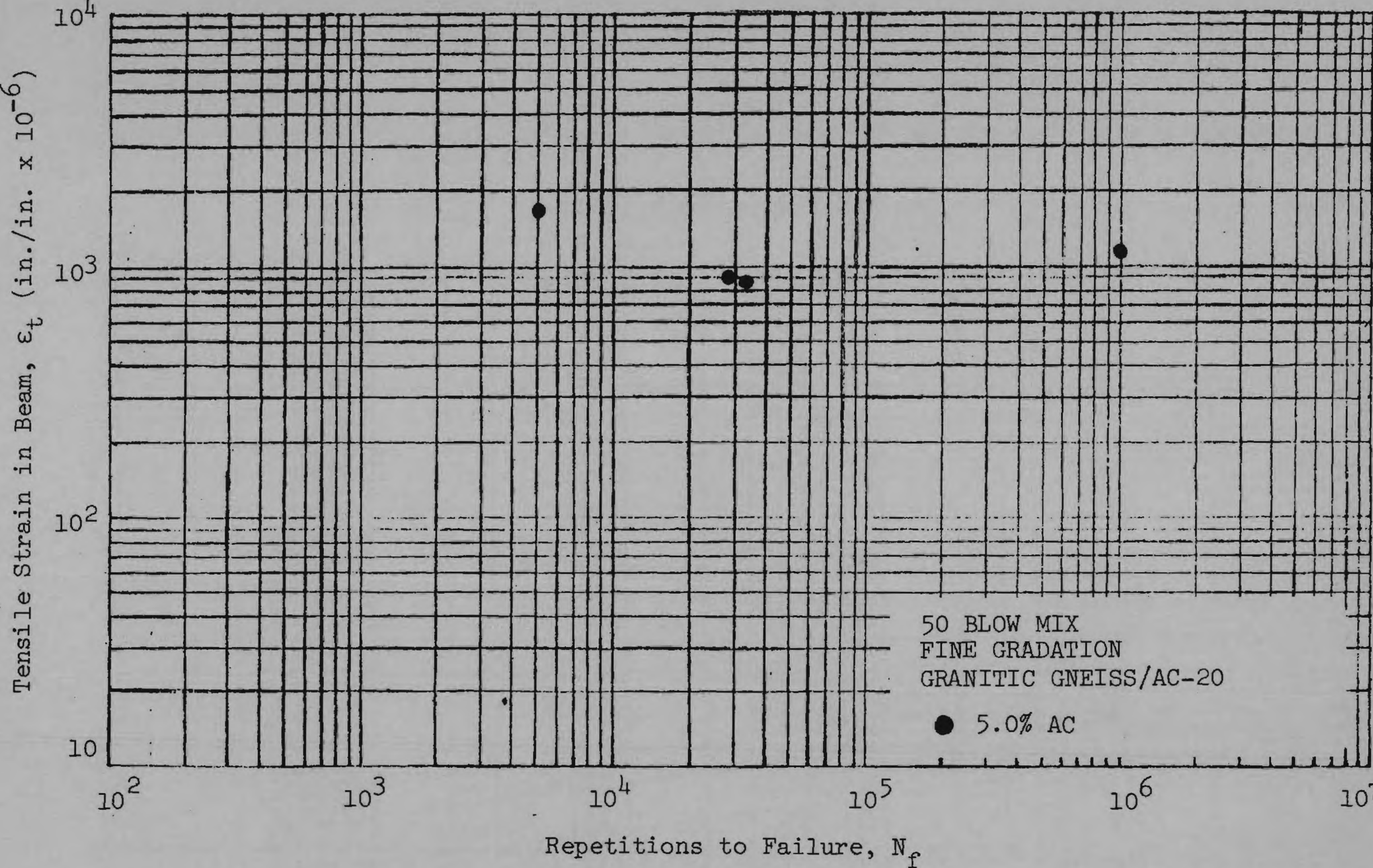


FIGURE C-14. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW BLACK BASE MIX WITH FINE GRADATION.

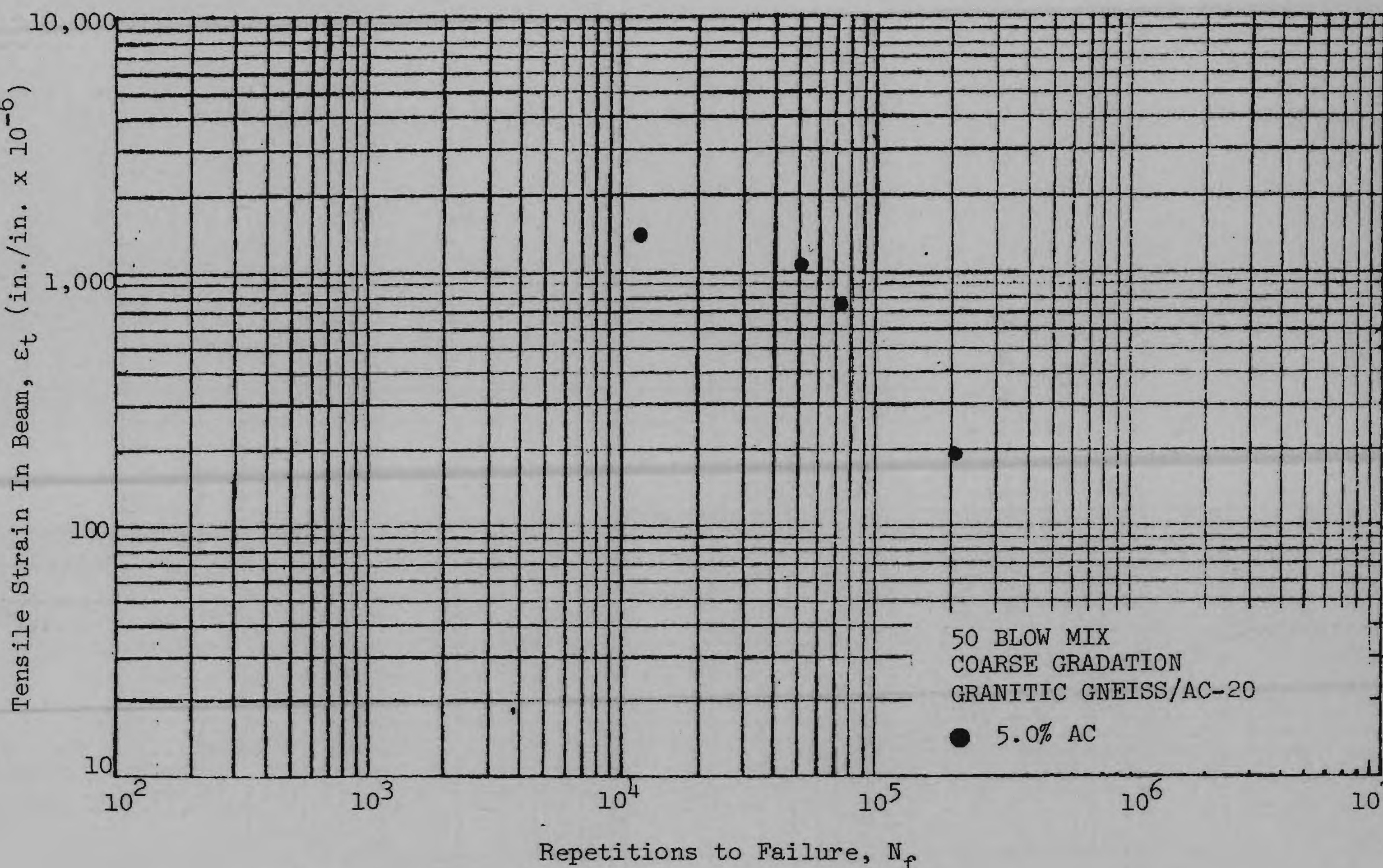


FIGURE C-15. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW BLACK BASE MIX WITH COARSE GRADATION.

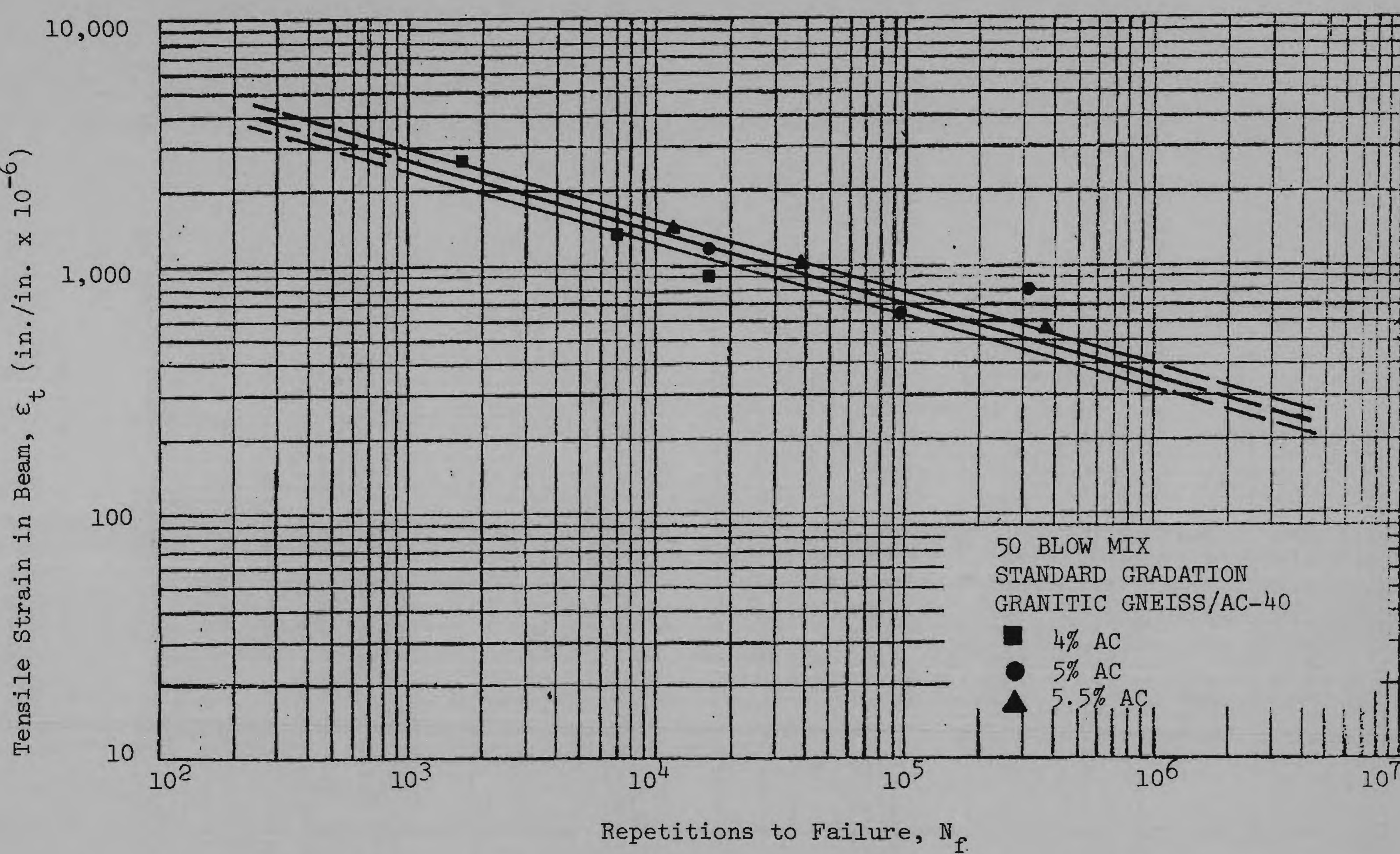


FIGURE C-16. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW BLACK BASE MIX WITH AC-40.

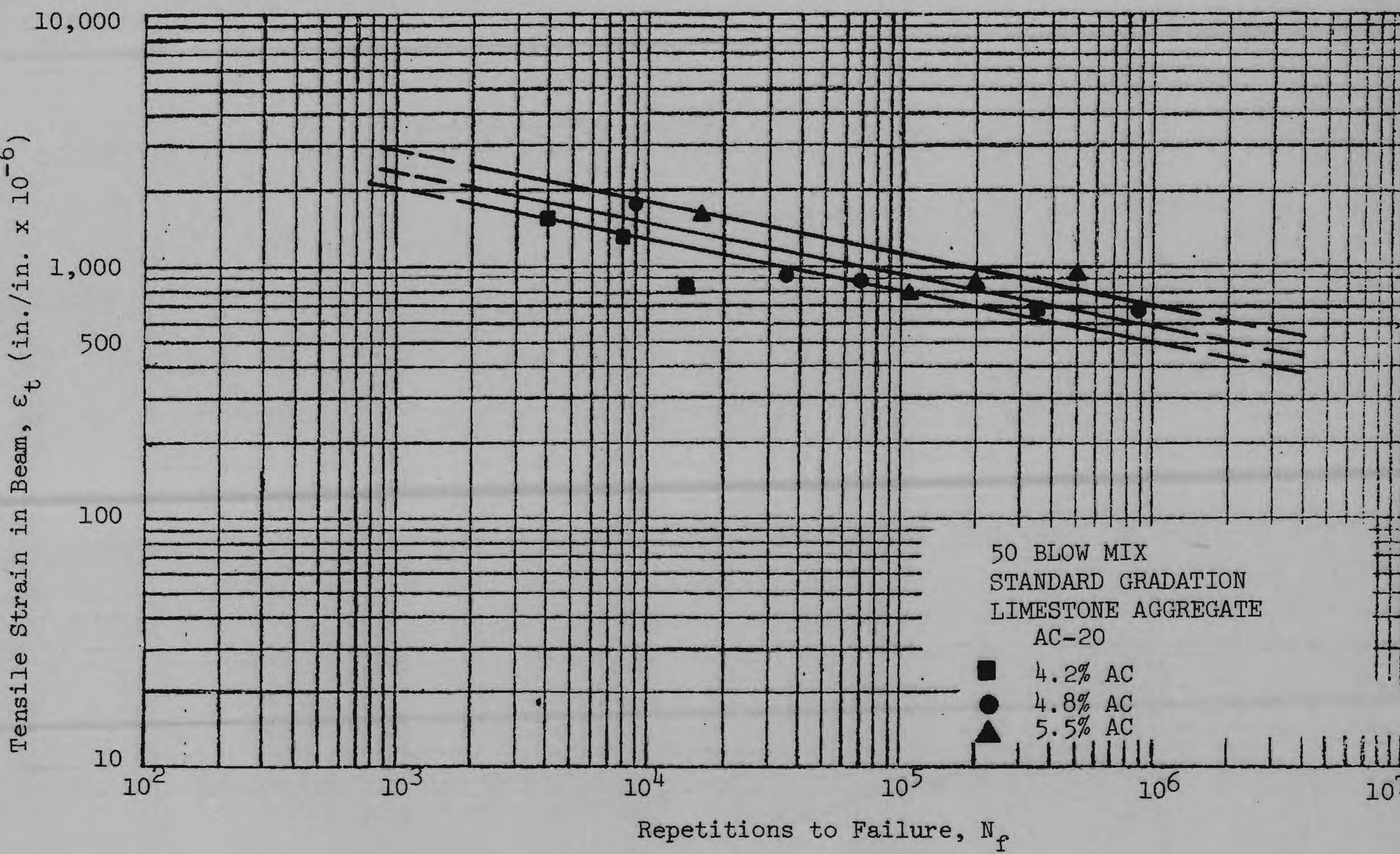


FIGURE C-17. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW BLACK BASE MIX WITH LIMESTONE AGGREGATE.

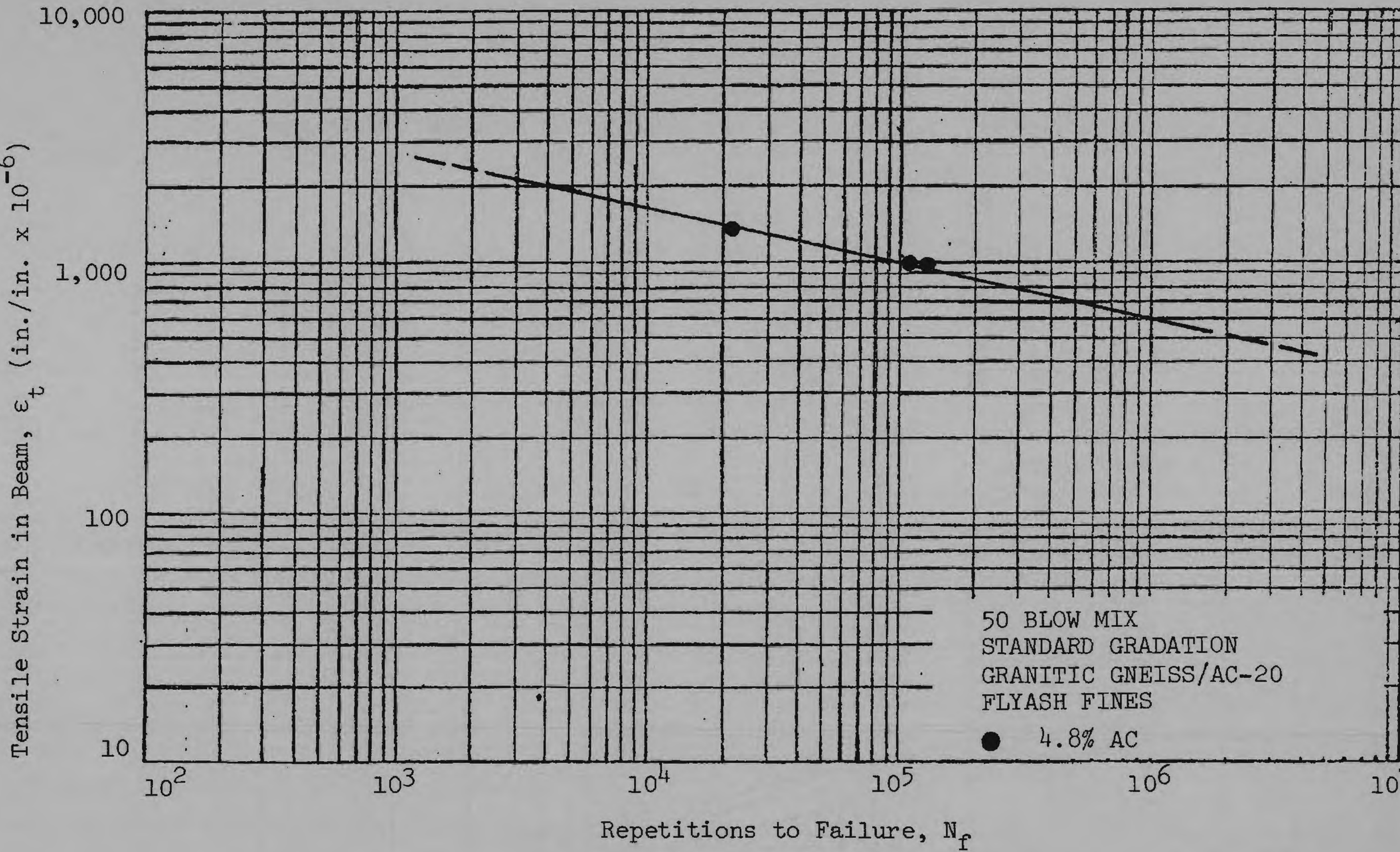


FIGURE C-18. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW BLACK BASE MIX WITH FLYASH MINERAL FILLER.

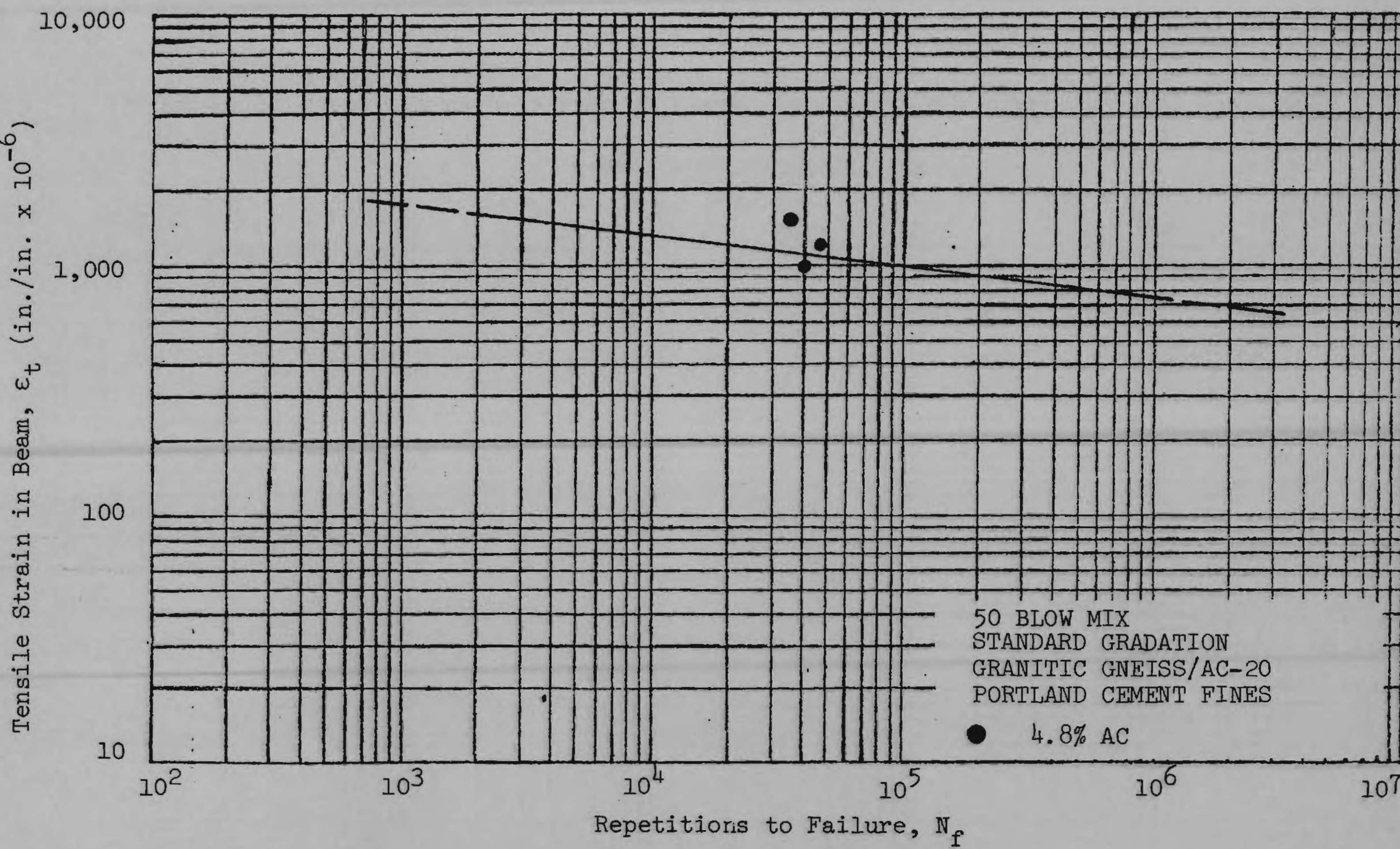


FIGURE C-19. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW BLACK BASE MIX WITH PORTLAND CEMENT MINERAL FILLER.

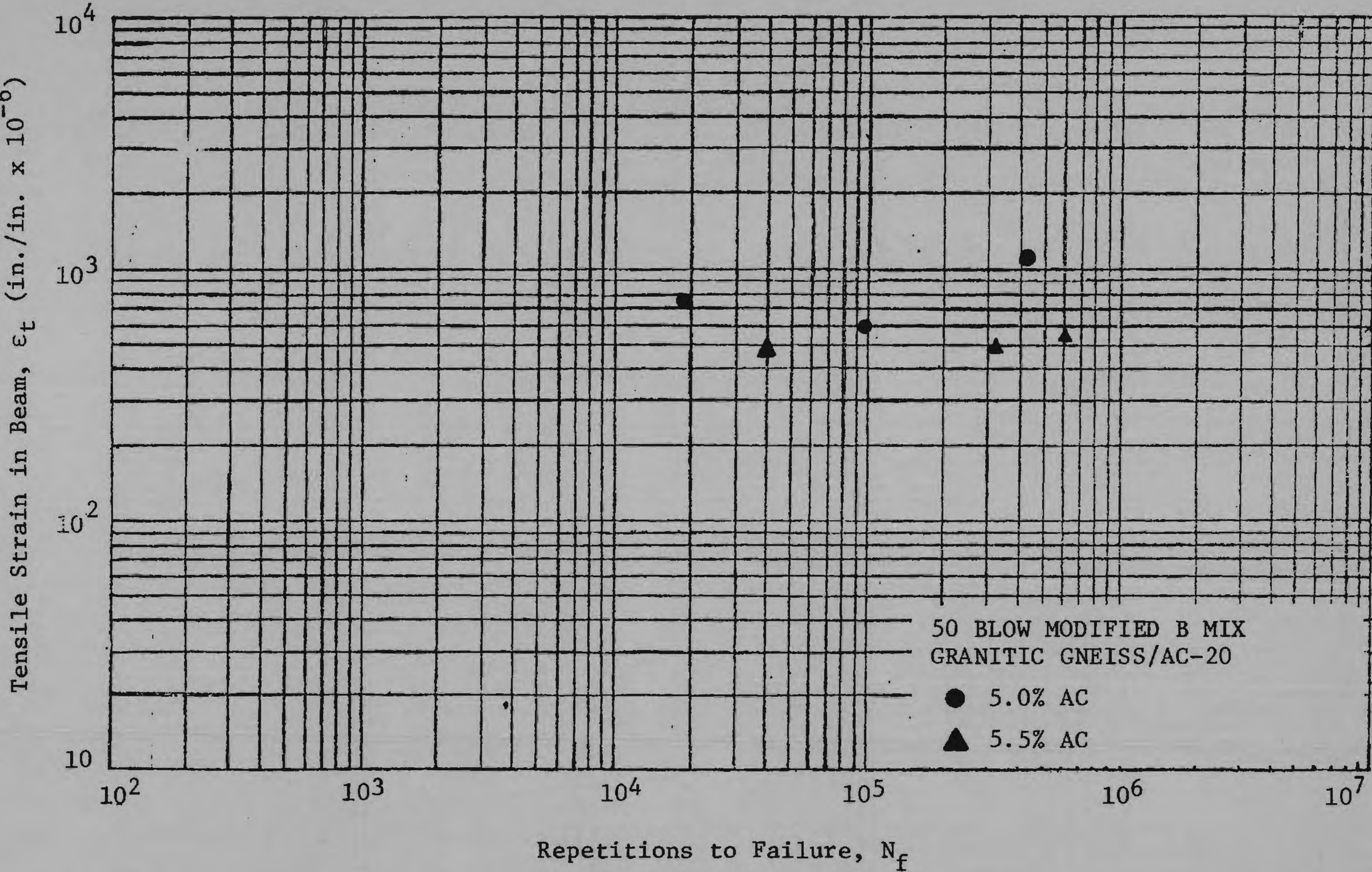


FIGURE C-20. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 50 BLOW MODIFIED B MARSHALL MIX.

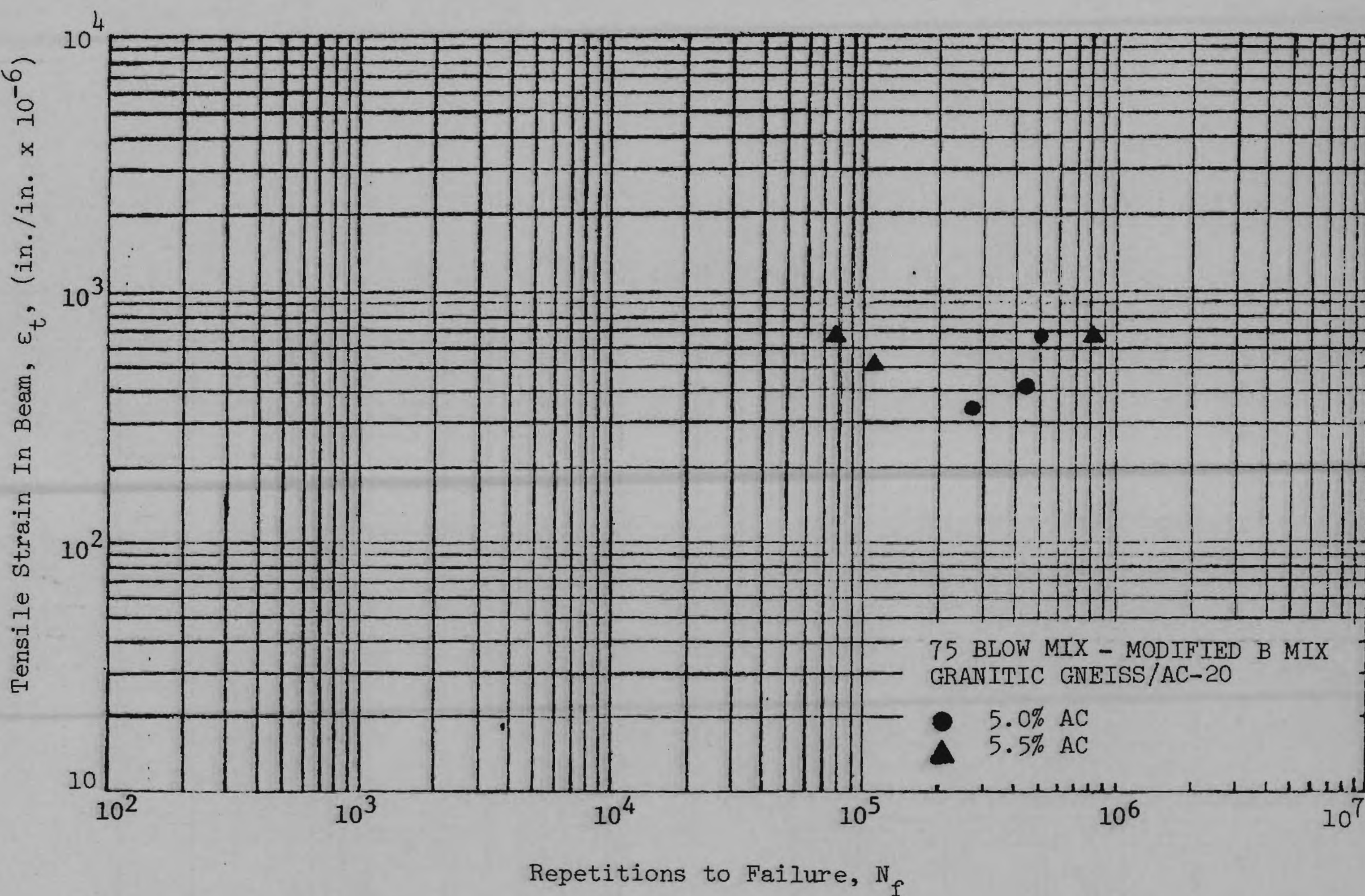


FIGURE C-21. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - 75 BLOW MODIFIED B MIX.

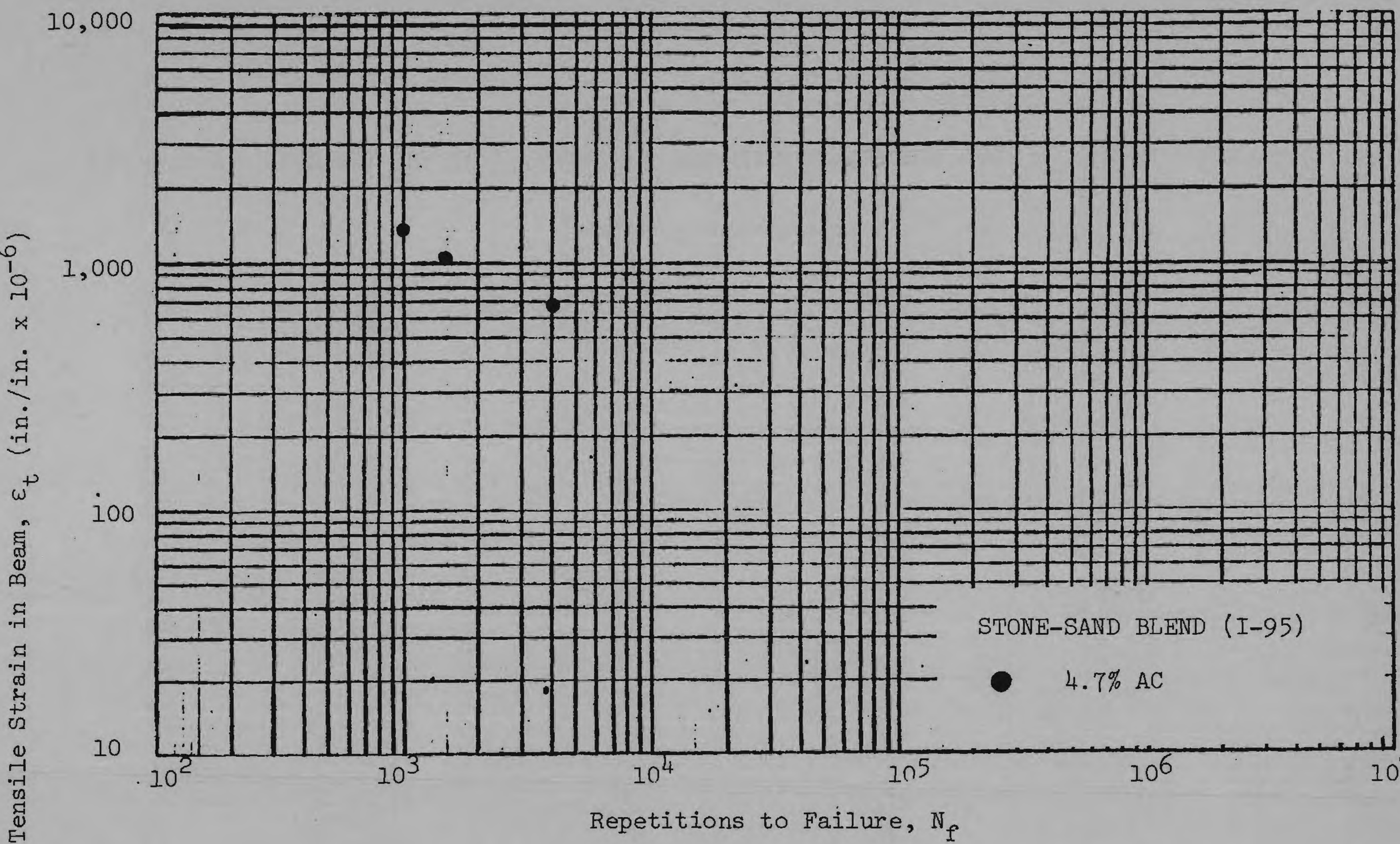


FIGURE C-22. TENSILE STRAIN IN BEAM AS A FUNCTION OF FATIGUE LIFE - STONE-SAND MIX (I-95)

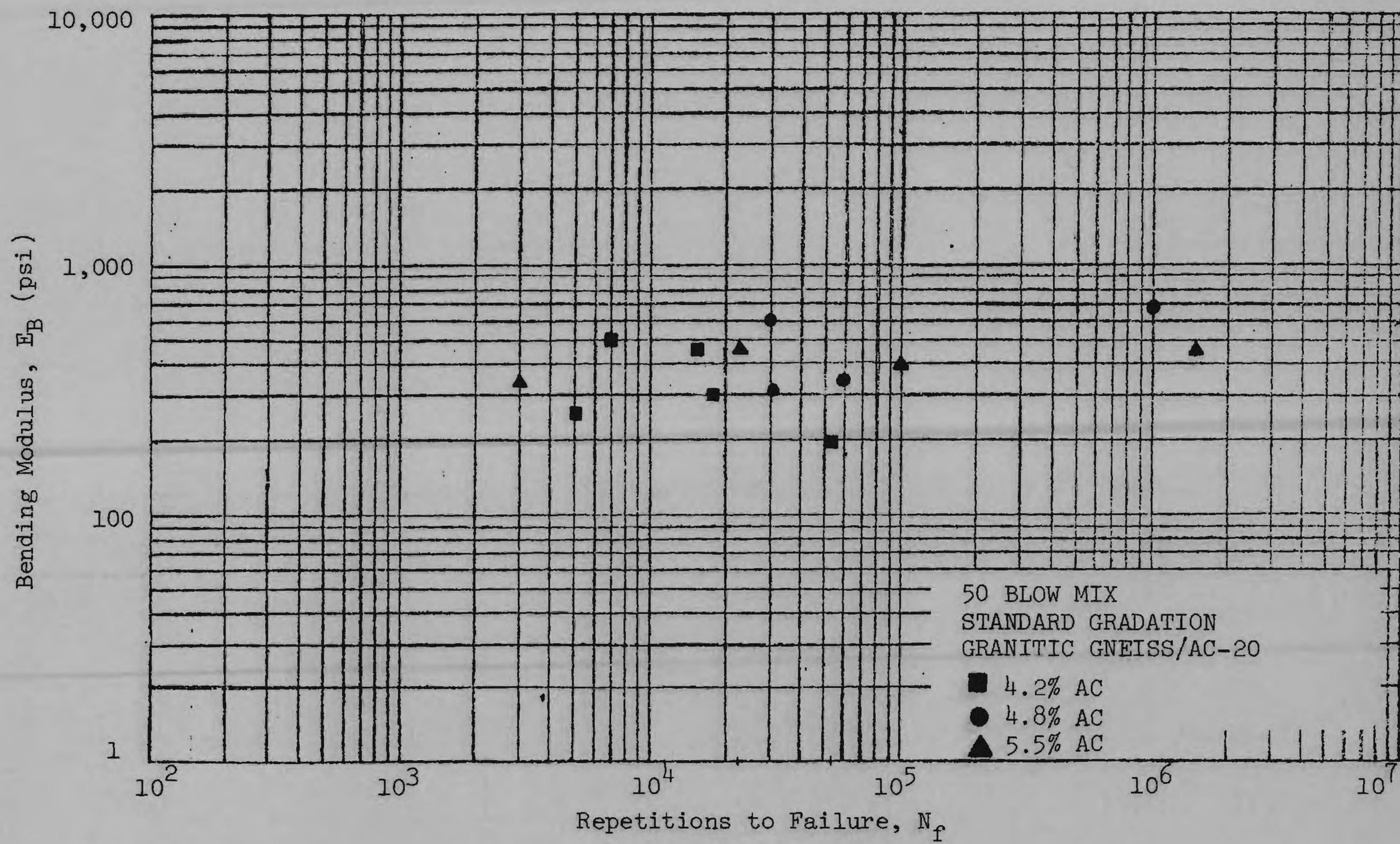


FIGURE C-23. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW BLACK BASE MIX.

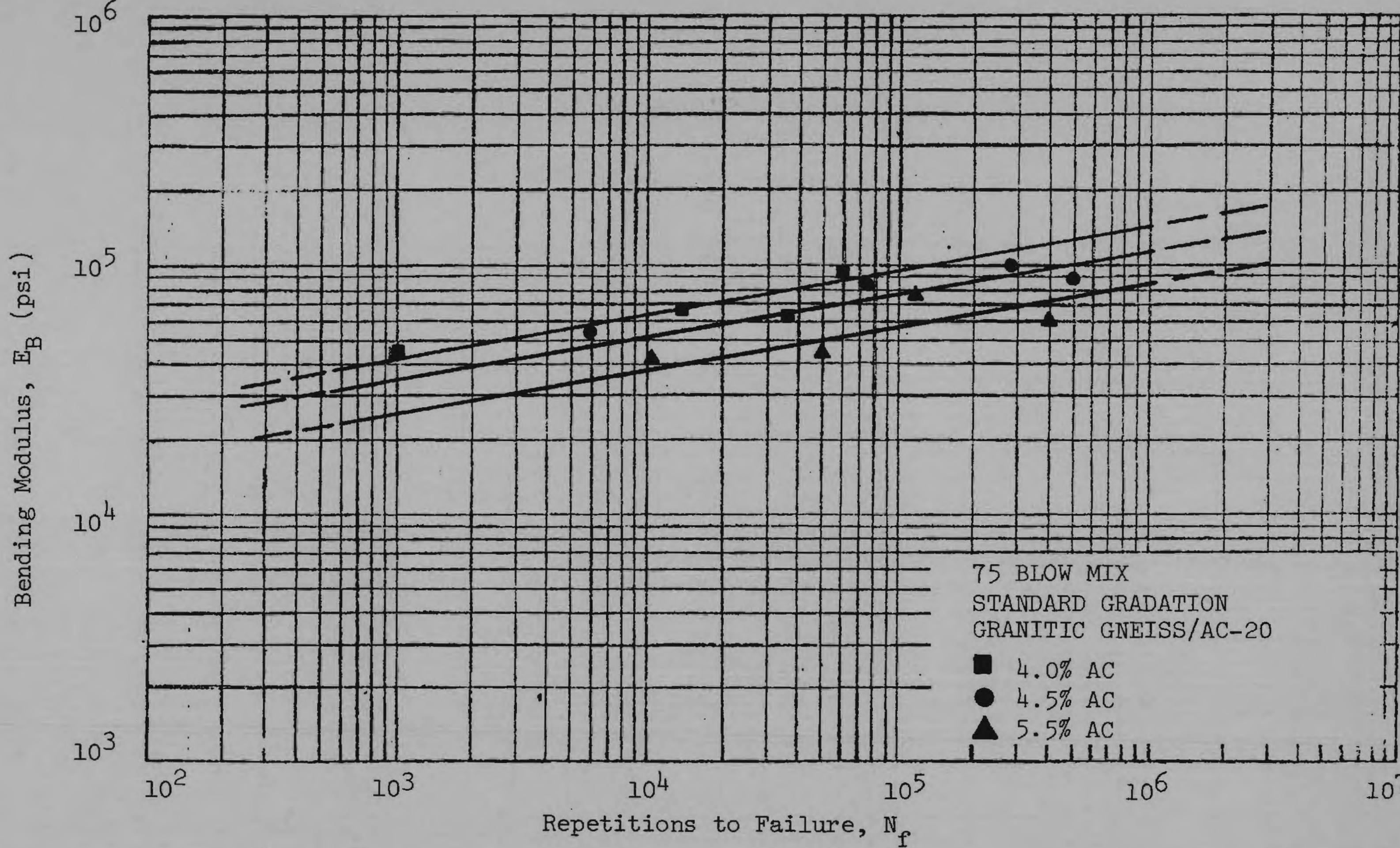


FIGURE C-24 EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 75 BLOW BLACK BASE MIX.

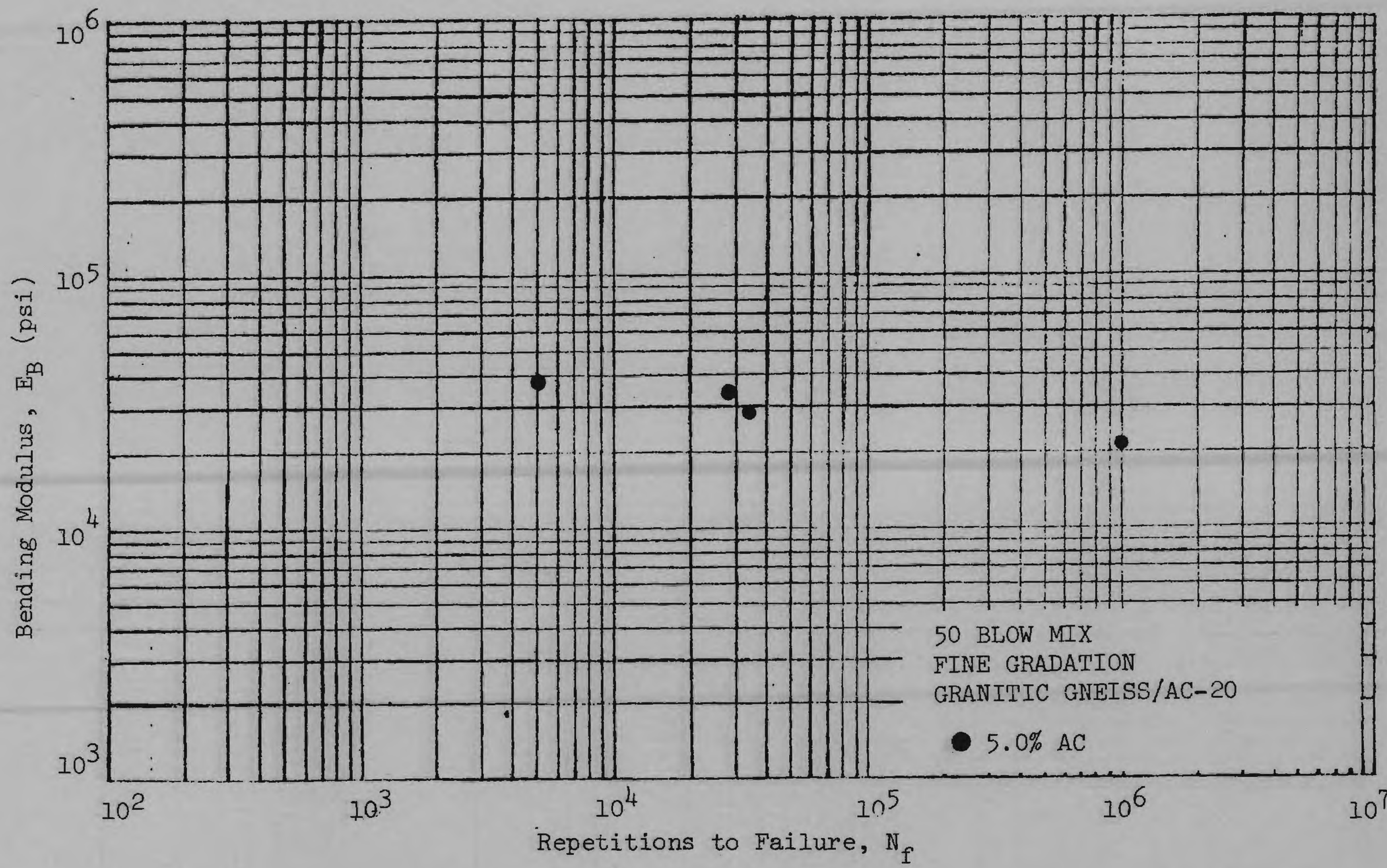


FIGURE C-25. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW BLACK BASE MIX WITH FINE GRADATION.

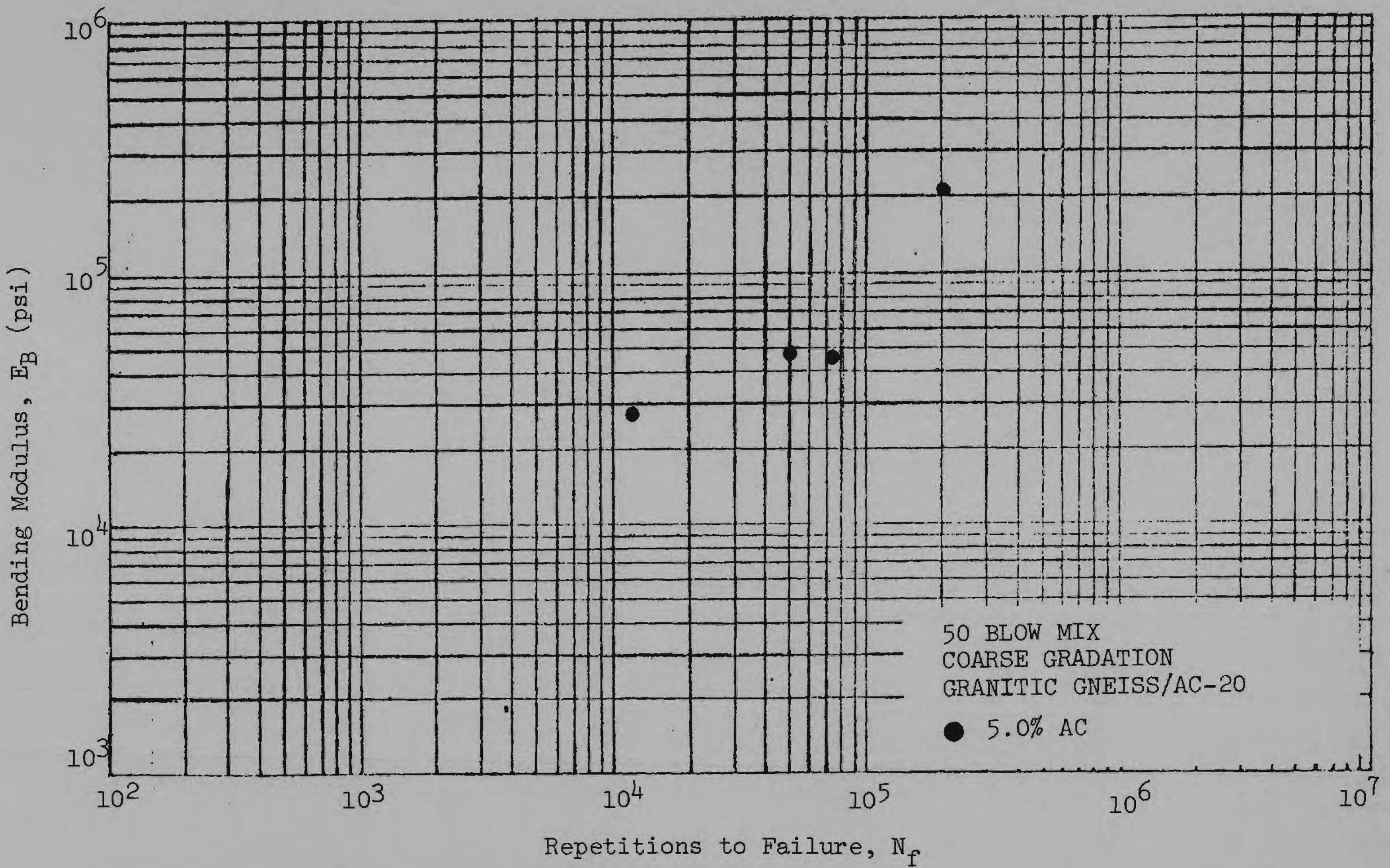


FIGURE C-26. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW BLACK BASE MIX WITH COARSE GRADATION.

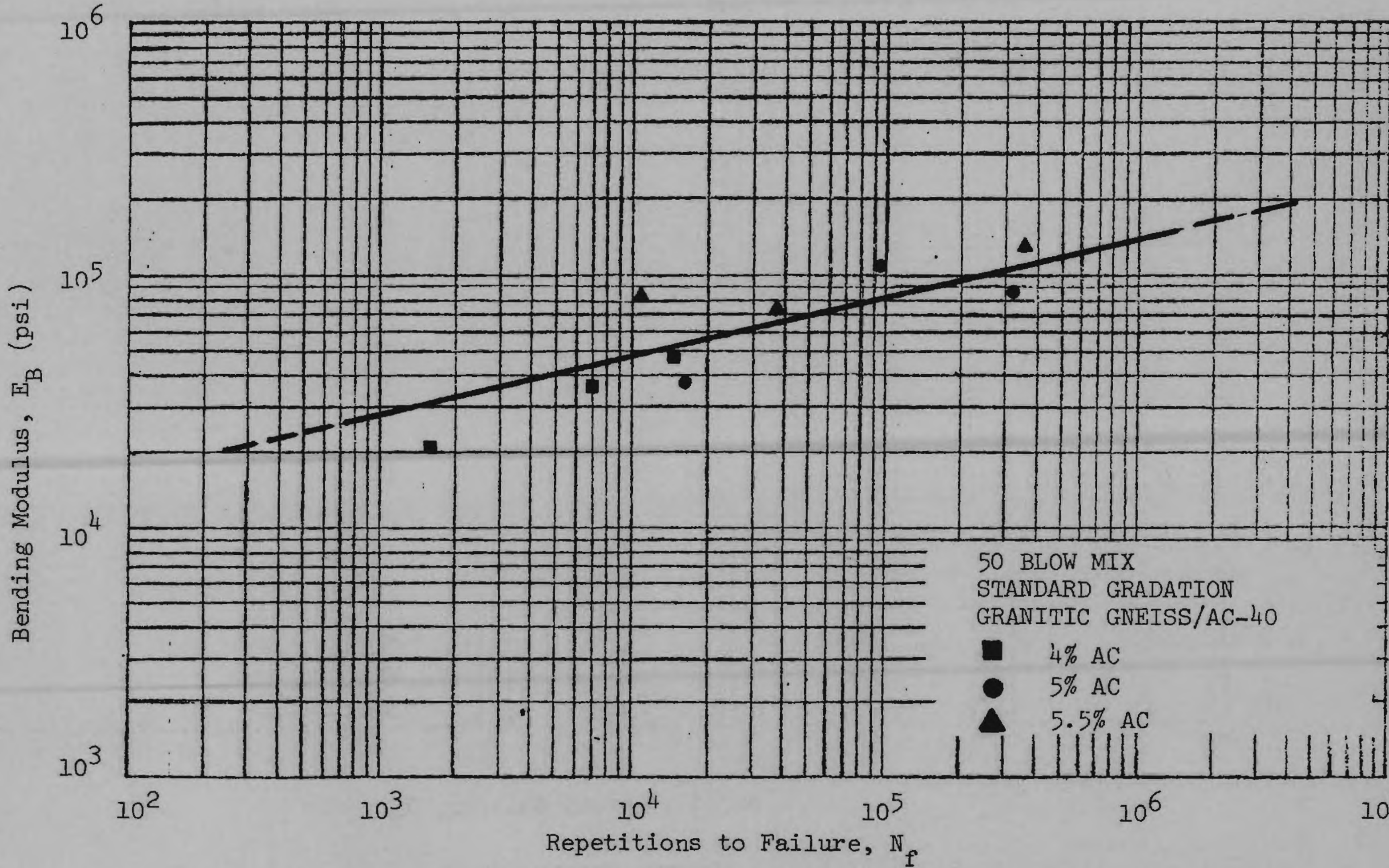


FIGURE C-27. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW BLACK BASE MIX WITH AC-40.

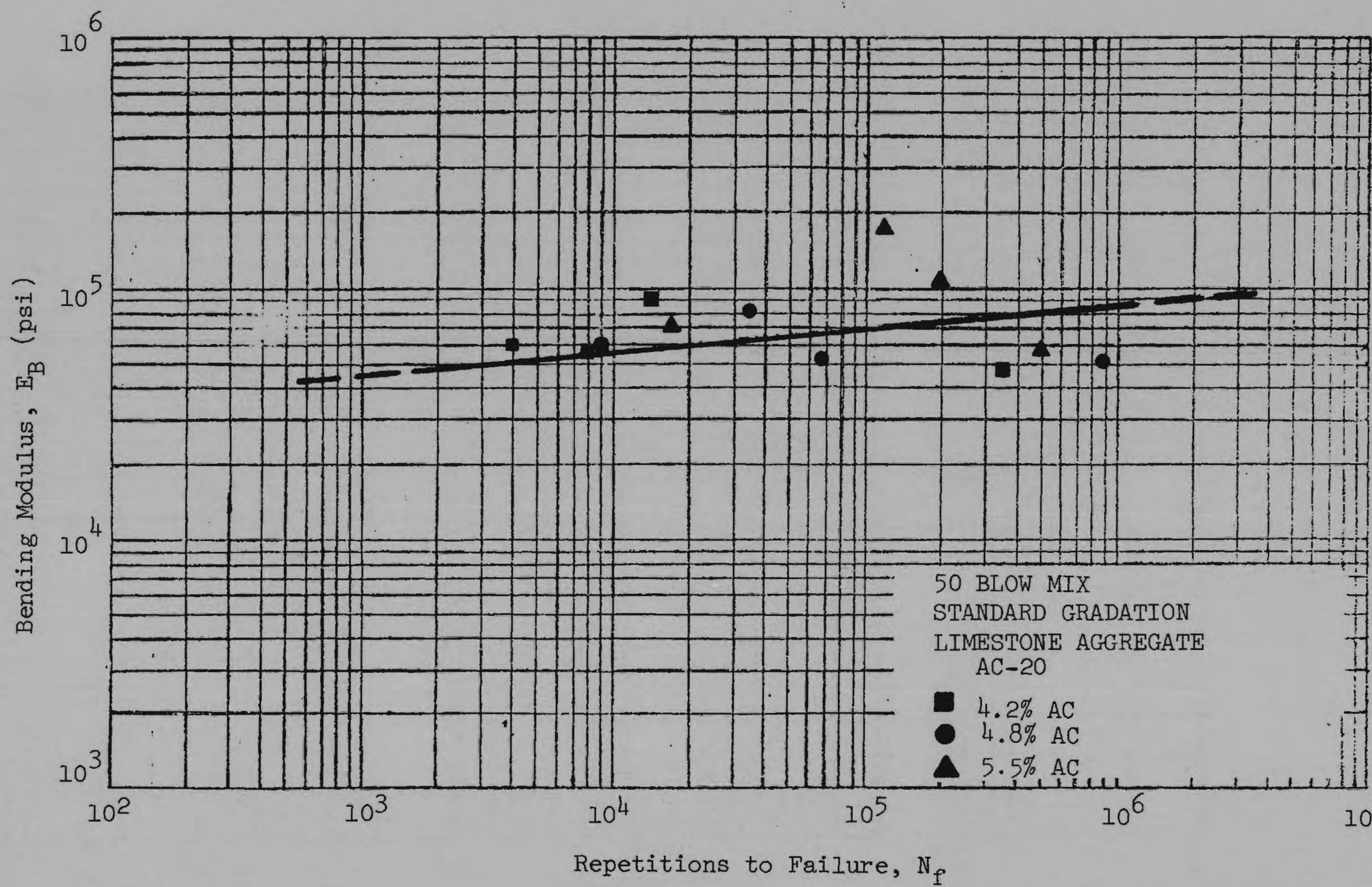


FIGURE C-28. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW BLACK BASE MIX WITH LIMESTONE AGGREGATE.

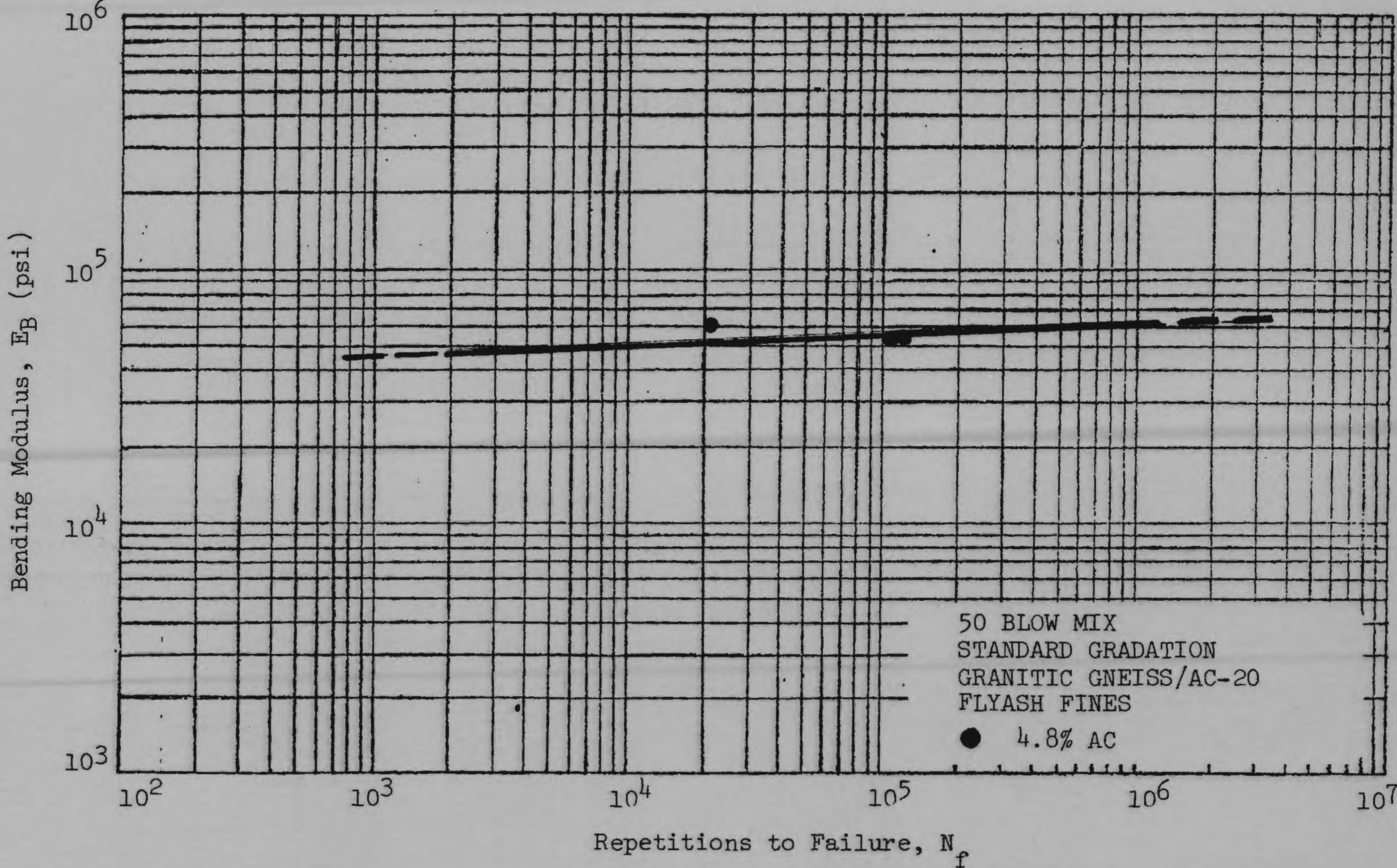


FIGURE C-29. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW BLACK BASE MIX WITH FLYASH MINERAL FILLER.

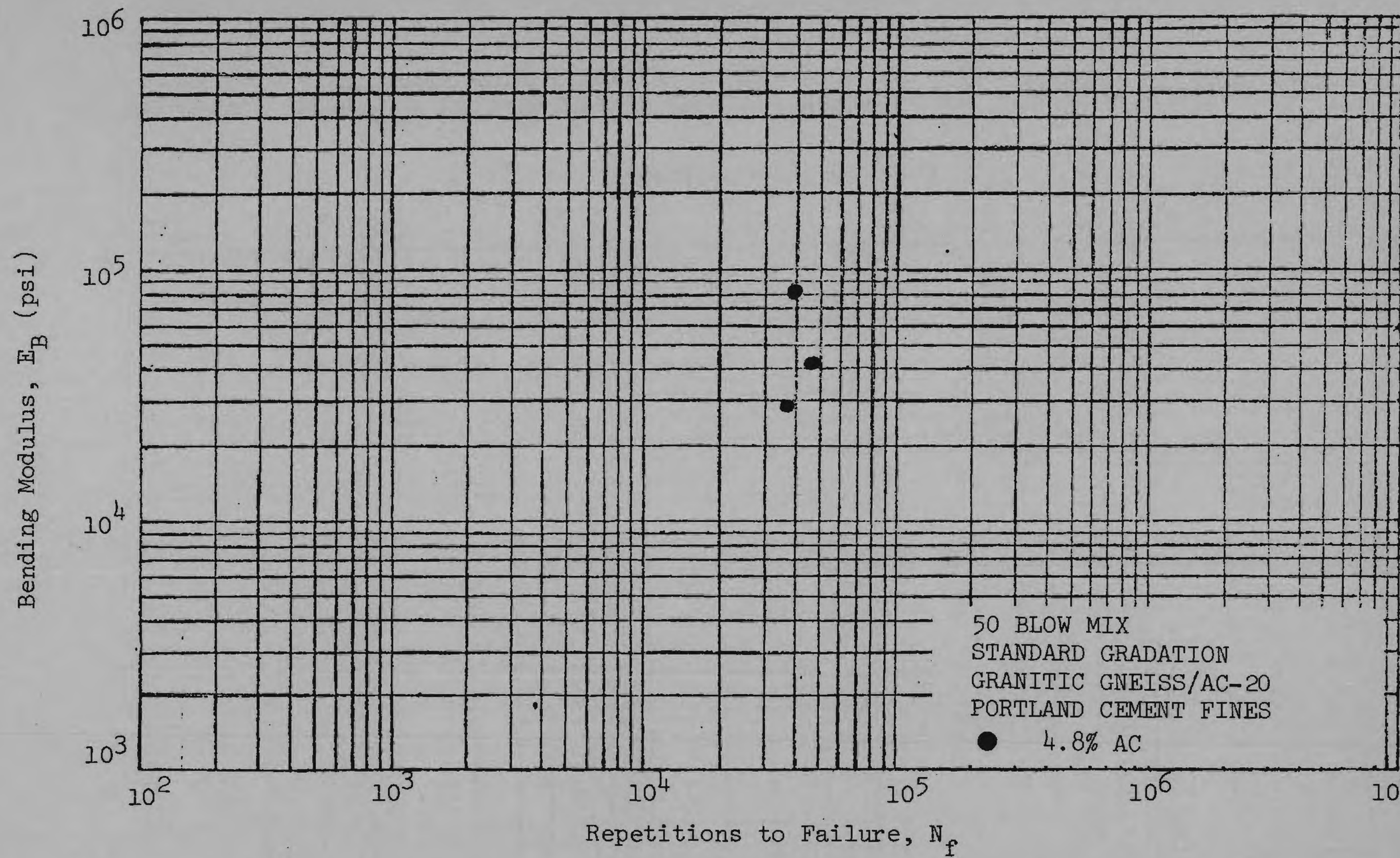


FIGURE C-30. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW BLACK BASE MIX WITH PORTLAND CEMENT MINERAL FILLER.

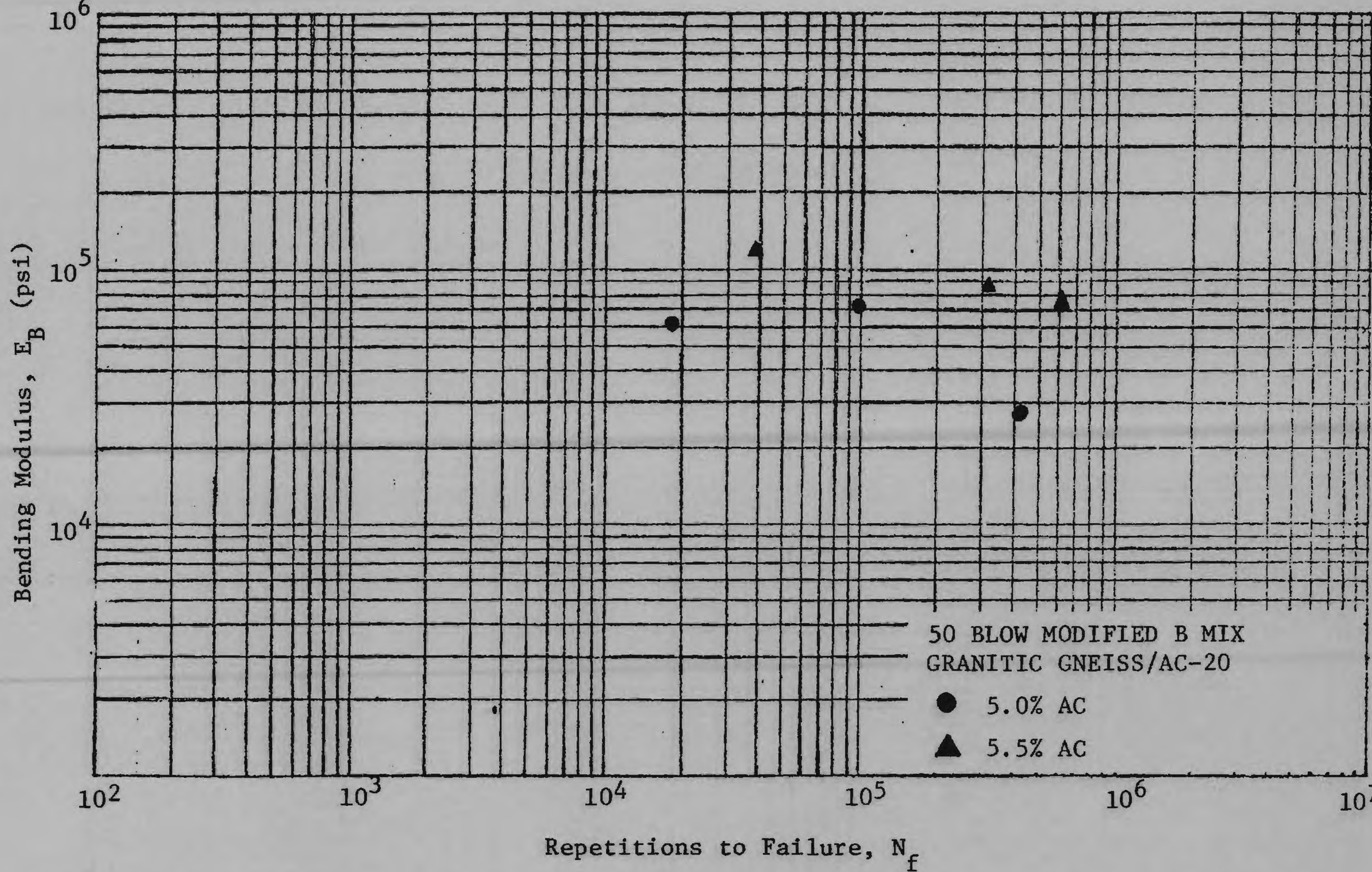


FIGURE C-31. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 50 BLOW MODIFIED B MARSHALL MIX.

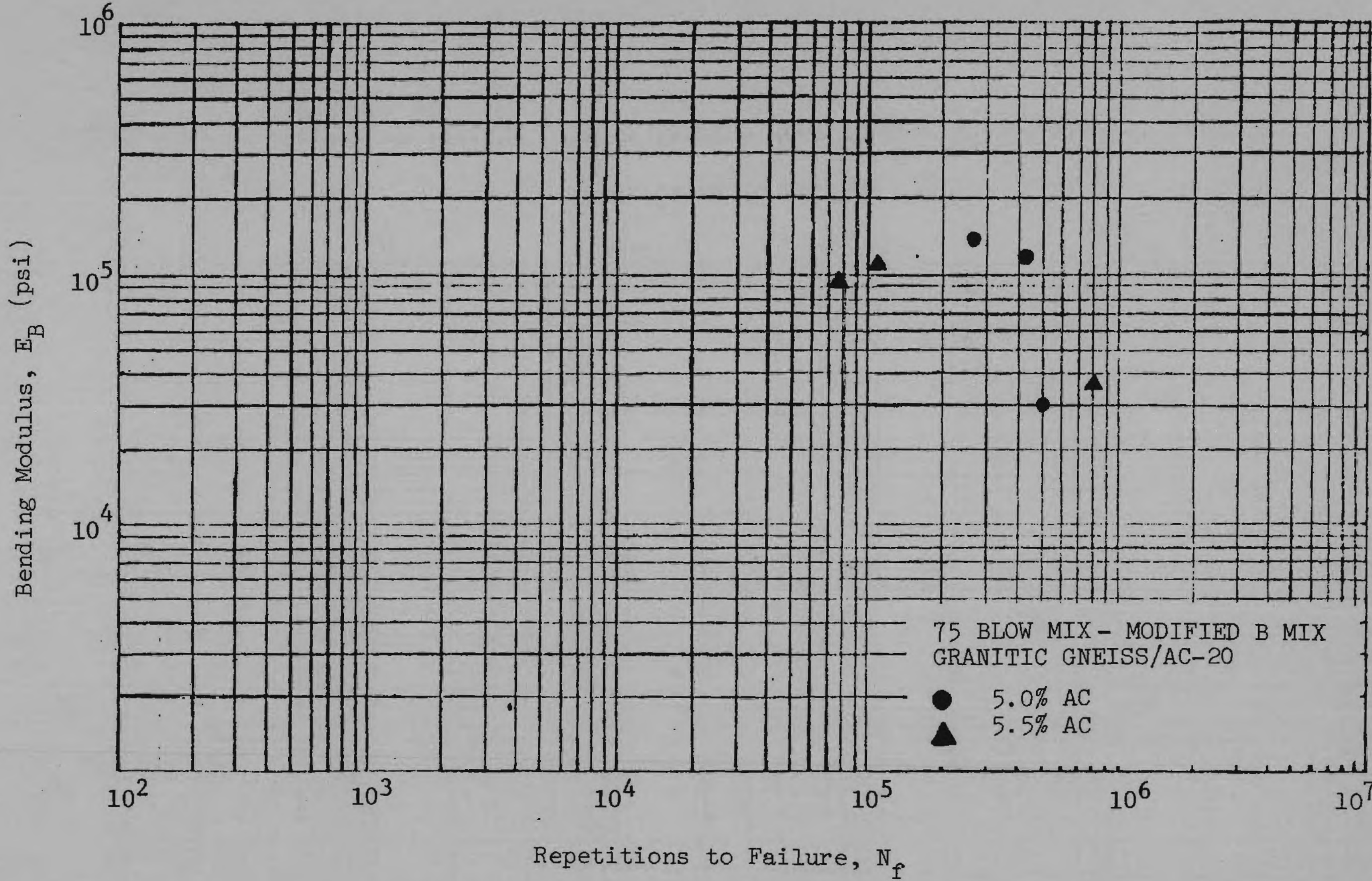


FIGURE C-32. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - 75 BLOW MODIFIED B MIX.

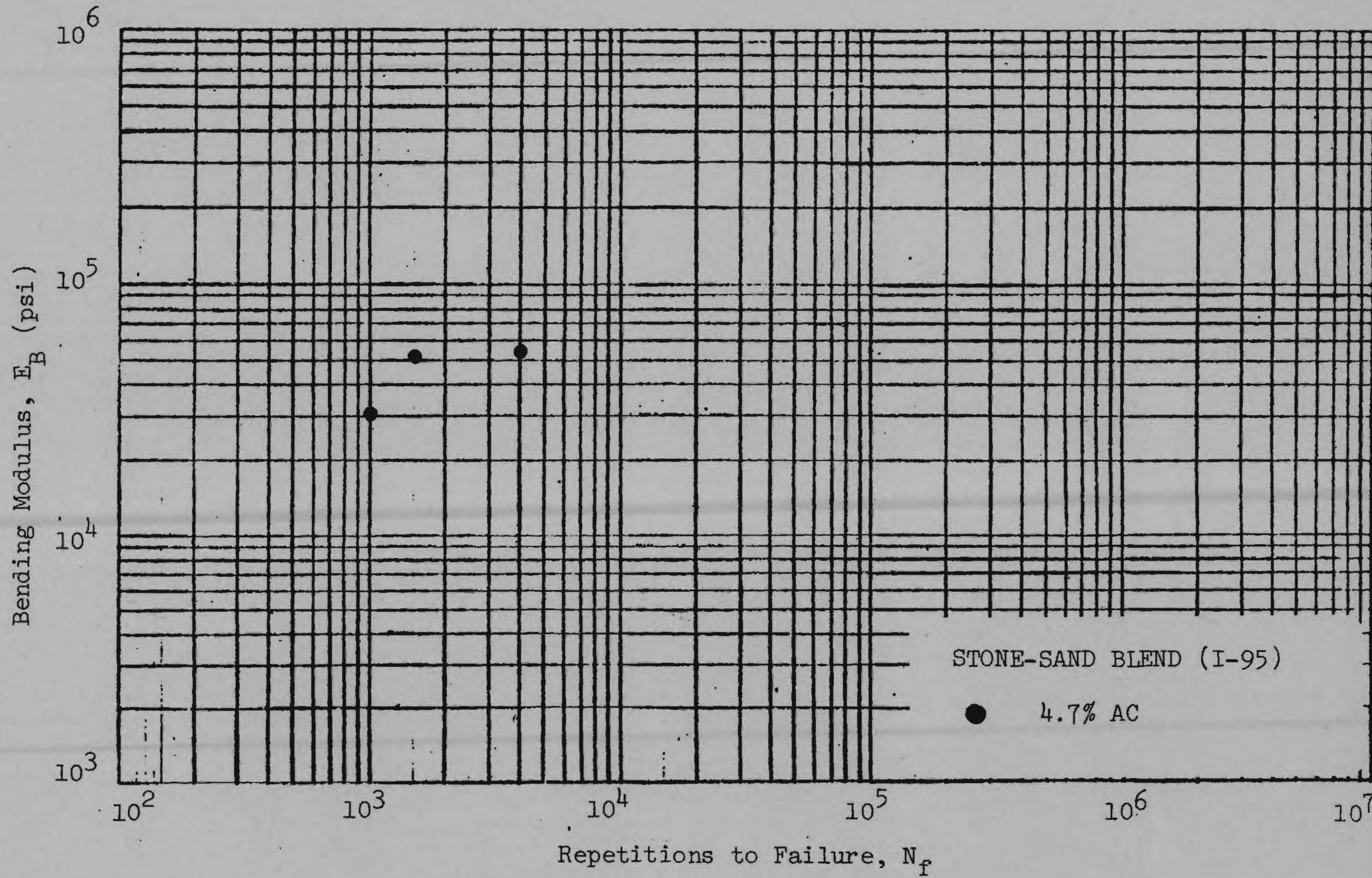


FIGURE C-33. EFFECT OF FATIGUE LIFE ON BENDING MODULUS OF MIX - STONE-SAND MIX (I-95).