AN EVALUATION OF FACTORS AFFECTING THE TENSILE PROPERTIES OF ASPHALT-TREATED MATERIALS

by

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Evaluation of Tensile Properties of Subbases
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PREFACE

This is the second in the series of reports dealing with the findings of the research project concerned with the evaluation of the properties of stabilized subbase materials. This report is intended to present some of the factors which are important in determining the strength of asphalt-treated materials and to report the findings of an evaluation by indirect tensile test of eight factors which were thought to affect the tensile properties of asphalt-treated materials. The report summarizes the effects of these eight factors and their interactions on tensile properties as well as the statistical design and analysis used in the evaluation.

The culmination of this report required the assistance of many individuals. The authors would like to acknowledge some of the people who contributed to this report. Special thanks are extended to Dr. Virgil L. Anderson for his help in designing the statistical experiment and providing guidance in the analysis of the data. Special appreciation is due Messrs. Pat Hardeman and Jim Anagnos for their assistance in the preparation and testing of the asphalt-treated materials. Thanks are also due to Mr. James L. Brown of the Texas Highway Department who provided the technical liaison for the project.

Future reports are planned which will be concerned with the tensile characteristics and behavior of cement-treated and lime-treated materials. Reports will be written on subjects such as (1) factors affecting the tensile characteristics and behavior of all three materials when subjected to static loads and dynamic repeated loads, (2) correlation of indirect tensile test parameters with parameters from standard Texas Highway Department Tests, (3) performance criteria for stabilized materials, (4) feasibility of determining an effective modulus of elasticity and Poisson's ratio from results of indirect tensile test, and (5) development of support value k for a layered system related to layer thickness, modulus, and the area of loading.

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LIST OF REPORTS

Report No. 98-1, "An Indirect Tensile Test for Stabilized Materials," by W. Ronald Hudson and Thomas W. Kennedy, summarizes current knowledge of the indirect tensile test, reports findings of a limited evaluation of the test, and describes the equipment and testing techniques developed.

Report No. 98-2, "An Evaluation of Factors Affecting the Tensile Properties of Asphalt-Treated Materials," by William O. Hadley, W. Ronald Hudson, and Thomas W. Kennedy, discusses factors important in determining the tensile strength of asphalt-treated materials and reports findings of an evaluation of eight of these factors.

Report No. 98-3, "Evaluation of Factors Affecting the Tensile Properties of Cement-Treated Materials," by Humberto J. Pendola, Thomas W. Kennedy, and W. Ronald Hudson, presents factors important in determining the strength of cement-treated materials and reports findings of an evaluation by indirect tensile test of nine factors thought to affect the tensile properties of cement-treated materials.

ABSTRACT

The indirect tensile test was used to evaluate factors which affect the tensile properties of asphalt-treated materials. Eight factors were evaluated at two levels in a 1/4 replicate of a full factorial statistical design. The factors investigated included aggregate type, aggregate gradation, asphalt viscosity, asphalt content, compaction type, mixing temperature, compaction temperature, and curing temperature. The test parameters used as indicators of the materials' tensile properties were indirect tensile strength and horizontal failure deformation.

An analysis of variance was conducted to determine the significance of the effects of all main factors, two-way interactions, and certain three-way interactions on the test parameters. Tables showing the order of significance as well as plots indicating the effects of those factors and interactions significant at an alpha level of 0.01 are presented in the report. A regression analysis was also conducted on those factors and interactions significant at an alpha level of 0.05 or greater to obtain predictive equations for both of the test parameters. These regression equations along with their regression coefficients and standard errors of estimate are presented in the report.

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CHAPTER 1. INTRODUCTION

The widespread interest in and demand for better highways has caused the professional highway engineer to reexamine the validity of current pavement design methods. In the past, empirical design methods have adequately served the highway needs, but the weaknesses in these methods have been revealed by the combination of heavy wheel loads and large traffic volumes prevalent today (Ref 1).

In an effort to provide adequate highways for these increased traffic volumes and wheel loads, there has been an increase in the use of unconventional road structures utilizing greater thicknesses of asphalt layers as well as combinations of rigid slabs with flexible bases and subbases.

The structure composed wholly of thicker asphalt layers can be analyzed as a system of elastic layers, bonded together perfectly, resting on a semi-infinite elastic mass (Ref 2). The composite structure of a slab on a stabilized base or subbase can be analyzed using both rigid pavement and layered system techniques. The rigid pavement analysis involves a slab-on-foundation similar to that developed by Westergaard (Refs 3 and 4) or by Hudson and Matlock (Ref 5). In this method the subbase is portrayed as a Winkler foundation consisting of a bed of elastic springs. The spring constant or modulus of subgrade support used in the rigid analysis can be obtained from a layered system analysis of the base, subbase, and subgrade materials underneath the slab or from field measurements such as plate load tests.

In either design concept, a better understanding of the contribution of the different layers to the behavior of the pavement structure as a whole is required. One important parameter used to describe the properties of the individual layers is flexural or tensile strength.

Tensile stresses are created in the individual layers of the roadway structure by moving traffic. As a vehicle moves along the highway, the layers of the pavement structure deflect under the weight of the vehicle, creating tensile stresses in the underside of each of the layers (Refs 2 and 6). If the stresses in the layers exceed the tensile strength of the material, the layer will crack, leading to eventual failure of the pavement structure.

Unfortunately, information on the tensile properties of different asphalt-treated materials has not been available, due to the lack of a satisfactory tensile test. However, this deficiency appears to have been alleviated by the use of the indirect tensile test in evaluating stabilized materials at The University of Texas (Ref 7).

From a review of the literature, it appears that many factors affect the properties of asphalt-treated materials. Previous studies conducted to evaluate these properties have usually limited the number of factors investigated at one time to two or three while holding constant any additional factors which might affect the material properties. Such studies can not give consideration to the interaction* effects of those factors held constant with those being investigated.

The experiment discussed herein is designed to establish the factors and interactions significantly affecting the tensile properties of asphalt-treated materials by investigating a large number of factors at two different levels in the same experiment. In order to accomplish this without an exorbitant number of test units, fractional factorial concepts will be employed. This approach is relatively new to the study of factors affecting tensile properties of asphalt-treated materials. The results of this study can be used as a screening experiment to establish those significant factors and interactions which may need investigating in greater detail.

*Interaction is the failure of two or more factors to act independently of each other; i.e., the response with respect to one factor is dependent upon the level of magnitude of one or more additional factors.

CHAPTER 2. SOME CONSIDERATIONS OF FACTORS AFFECTING STRENGTH OF ASPHALT-TREATED MIXTURES

The stability or strength of an asphalt mix is dependent upon the two major components, the asphalt and the mineral matter. When subjected to stress, asphalt-treated mixtures may exhibit flow properties of the asphalt or unyielding properties of the aggregate, depending upon the different temperature and loading conditions applied. A discussion explaining these phenomena is presented later in this chapter. Table 1 presents a general stability chart of the mixture under various mix, loading, and environmental conditions. The extreme values shown in the table are compared to an ideal mixture with optimum asphalt content.

From the table it can be seen that asphalt contents greater than the optimum amount tend to produce instability, while asphalt contents less than the optimum produce a stable mix. It is also apparent that high pavement temperature, slow traffic speed, and polished aggregate also tend to make an unstable asphalt mixture.

A disturbing factor in the current design of asphalt-treated mixtures is the fact that different optimum design values are reported for similar projects, depending upon the type of test and design method used (Ref 10). However, in recent years there have been increased efforts to investigate asphalt stabilization on a more scientific basis and to produce a rational method of design with a theoretical background which could be universally used (Refs 11, 12, and 13). McLeod (Ref 13) defines the rational approach to the design of asphalt pavements as a method for determining or expressing their strength or stability in terms of pounds per square inch or some other unit stress basis, similar to those employed for indicating the strength of steel, concrete, etc.

There is presently under development at The University of Texas a design method based upon the indirect tensile test (Refs 14 and 17). This test was developed independently by Akazawa (Ref 15) of Japan and Carneiro (Ref 16) of Brazil in 1953. A compressive load applied to a cylindrical specimen along two opposite generators produces a uniform tensile stress over the diametrical plane containing the applied load. The tensile strength of the specimen is

TABLE 1. AN ANALYSIS OF STABILITY OF ASPHALT-TREATED MIXTURES (from Ref 8)

Yielding	Mix and Environmental Condition		Unyielding
Unstable properties of aggregate and yielding properties of asphalt tend to predominate	Optimum asphalt content		Unyielding properties of asphalt and stable properties of aggregate tend to predominate
	More asphalt	Less asphalt	
	Pavement temperature		
	Hot	Cold	
	Traffic speed		
	Slow	Fast	
	Aggregate type		
	Polished rounded	Rough angular	

related to the maximum load and is expressed in terms of pounds per square inch. The equations for stresses at any point in a diametrically loaded disk have been developed by Frocht (Ref 17), A. and L. Föppl (Ref 18), Peltier (Ref 19), Ramesh and Chopra (Ref 20), Timoshenko and Goodier (Ref 21), Hertz (Ref 22), and Hondros (Ref 23).

An important factor which must be considered is the test method used to simulate actual field conditions. The test procedure used, because of its dependence upon standard loading rates and test temperatures, exerts a great influence upon the indicated strength of an asphalt-treated mixture.

Some of the limitations of standard test methods as well as the effects of material characteristics and environmental conditions on strength of asphalt-treated materials are discussed in the following paragraphs.

METHOD OF TEST

The differences in performance of asphalt-treated mixtures with varying temperature and traffic loading conditions cause the major difficulties encountered in attempting to simulate field conditions in laboratory tests. A test method utilizing a rapid loading rate will cause a viscous resistance to deformation and corresponds to rapidly moving traffic and would not be expected to provide an indication of the ability of the mixture to withstand a static load over a period of time. Because asphalt is a thermoplastic material, i.e., one which softens when heated, a test conducted at room temperature will produce a higher stability value than one at a higher temperature.

Field performance as well as laboratory test results is highly dependent on the rate of loading and temperature. The variety of test temperatures and loading rates used in some of the existing laboratory stability test methods can be seen in Table 2.

Because of temperature and loading rate difficulties, most design methods were developed empirically and were based primarily upon a soil classification system, a soil-strength test, or a combination of the two (Ref 10). The test adopted for the particular design method was then used to evaluate good and poor pavements in order to obtain a comparison of the test with field conditions. The primary limitation with this type of procedure is that its use is normally restricted to areas with similar materials, construction techniques, and environmental factors.

Some of the tests outlined in Table 2 have received acceptance nationwide and some are used worldwide. Neppe (Ref 10), however, points out that each test method makes certain assumptions and perhaps overaccentuates some physical characteristics at the expense of others. Of those tests listed in the table, only the cohesiometer provides some measure of the tensile strength of a material.

TABLE 2. AN ANALYSIS OF STABILITY TEST METHODS (from Ref 8)

Test	Specimen Temperature	Rate of Loading	Reading
Hubbard-Field	140^{o} F	2"/min	Maximum load
Florida Bearing	room	50 psi/min	Maximum load at 0.1
Modified Florida Bearing	140^{o} F	60 psi/min (.05"/min)	1/4" penetration or 3/4" cracking
Hveem Stabilometer	140^{o} F	.05"/min	Lateral at 400 psi vertical load
Hveem Cohesiometer	140^{o} F	1800 gm/min	Maximum load
Smith Triaxial	room	20 psi static	Lateral; ϕ and C calculated
Marshall	140^{o} F	2"/min	Maximum load

Some of the factors which are important in producing strong asphalt mixtures are given below:

(1) characteristics and gradation of aggregates,
(2) asphalt content,
(3) compactive effort,
(4) temperature,
(5) loading rate, and
(6) repeated loading.

These factors are discussed in the following paragraphs.

EFFECT OF VARIATION IN CHARACTERISTICS AND GRADATION OF AGGREGATES

The stability of asphalt-treated mixtures subjected to compression is attained primarily through the interlock (Ref 24) or friction developed between the aggregate particles. The aggregate particles in the mixture are held in position by the asphalt once it has hardened.

Monismith (Ref 25) advises that the following factors contribute to the interparticle friction:

(1) particle surface texture,
(2) particle shape or angularity,
(3) void ratio,
(4) particle size, and
(5) particle gradation.

Stanton and Hveem (Ref 26) and Hveem (Ref 27) state that the surface characteristic of the mineral aggregate is the most important single quality affecting the stability of an asphalt pavement and advocate the use of rough stone to maintain as much friction as possible between the particles. Griffin and Kallas (Ref 28) have shown that increased angularity and surface roughness of fine aggregate produces increased stability values, increased percent of voids in aggregate, and increased optimum asphalt contents. They found that the type and quantity of fine aggregate used in the mixture have pronounced effects on its test properties. Other investigations (Refs 29, 30, and 31) also demonstrate that angular, rough-faced aggregate produces more stability than round or angular smooth-faced aggregate. Monismith (Ref 32) has shown that rougher textured materials allow more asphalt to be incorporated in the mixture, thus resulting in an increased fatigue life.

Void ratio, or degree of packing, influences internal friction. The lower the void ratio, the greater will be the degree of packing for a given aggregate gradation and the greater the frictional resistance of the aggregate mass (Ref 25). It has been demonstrated that aggregate with a small amount of angularity and interparticle friction requires less applied force to rearrange and consolidate the particles into a dense mass (Ref 33). However, this type of aggregate produces a less stable mix than angular aggregate.

According to Monismith (Ref 25), investigators have indicated that particle size has little effect on interparticle friction. Some of the test methods such as the Hveem and Marshall methods limit the maximum sizes of aggregates used in the test procedure; therefore, the test specimen may not be indicative of the actual pavement.

The effect of particle gradation on the internal friction and stability of a mix is a controversial subject (Ref 34). Hubbard (Ref 35) has stated that a reasonable distribution of individual sizes is desirable from the standpoint of prevention of segregation preceding and during the mixing and laying operations. Vokac (Ref 36) advocates the use of gradations which are fairly symmetrical; that is, the largest fractions should be fairly uniform in size and should not be grouped at one end of a gradation analysis. On the other hand, Spielmann and Hughes (Ref 37) recommend the use of the maximum percentage of large material in order to give low percent of voids. Olmstead (Ref 38) is also in favor of gradations with a predominance of large material to ensure weather resistance, adequate stability under traffic loadings, and ease of construction and maintenance. Stanton and Hveem (Ref 26) and Hveem (Ref 27) believe that dense gradations lead to critical conditions in which the voids tend to become overfilled with asphalt, thereby creating an unstable mix.

EFFECT OF ASPHALT CONTENT

In general the strength of a stabilized mixture increases with the addition of asphalt until a maximum stability value is obtained. The asphalt content at the maximum stability value is generally considered to be the optimum asphalt content. The addition of asphalt in excess of the optimum content will decrease the strength of the mixture below that of the aggregate alone before the addition of asphalt.

The amount of asphalt in a mixture appears to have a great influence on the behavior of asphalt-treated materials subjected to repeated loading (Refs 37, 39). Asphalt type, also, appears to produce significant effects, especially when different test temperatures are considered. At low temperatures the specimens with more viscous asphalt cement displayed a longer fatigue life at the same stress level (Ref 39).

There is a fairly well-defined range of optimum asphalt content for use with the types and gradations of aggregates normally used. The recommended asphalt contents for hot mix asphalts normally range from 4.0 to 10.0 percent of the mixture by weight (Ref 40). It should be noted that excessive amounts of asphalt stabilizing agents (Ref 24) can cause a plastic condition with subsequent failure of the mixture.

EFFECT OF COMPACTIVE EFFORTS

One of the primary difficulties encountered in the simulation of field conditions with laboratory testing involves the use of a realistic compactive effort and type of compaction. A great deal of work has been directed toward the formation of test specimens which represent the mixture in the field. The major compaction methods used in laboratory testing are impact-compaction (Ref 41), static-compression (Ref 42), kneading-compactor (Ref 43), and the gyratory-shear-compaction method (Ref 44).

The impact-compaction method consists of the application of a specific number of blows to each face of a specimen with a compaction hammer using a free fall of 18 inches. For asphalt-treated materials the impact method normally used is the Marshall method. This method requires the application of 35, 50, or 75 blows to each face as specified by the design traffic category.

The process of compacting specimens by subjecting them to a static load which is built up slowly to some predetermined value and then released is referred to as the static-compression method.

Kneading compaction imparts a kneading action consolidation by a series of individual impressions made with a ram. At each application of the ram a certain pressure is applied subjecting the specimen to a kneading action without impact (Ref 45).

The gyratory-shear-compaction method consists of a shearing action imparted to a specimen by the gyratory motion of a steel mold while pressures are maintained at each end by loading plungers whose faces remain parallel to each other (Ref 46).

The static-compression and impact-compaction methods were two of the first procedures used to control the compaction of the different soil types. The two methods also provided a way to study the properties of soils compacted under a uniform compactive effort (Ref 47).

Observation of the results of the two methods indicated that compaction curves for specimens compacted by the impact method did not have the same characteristic shape as those exhibited by the static-compression method (Ref 47).

Studies also showed that the stress-strain characteristics of asphalt-paving mixtures compacted by the two methods were different (Ref 48).

Because of early observations, studies were made into the nature of the compactive effort produced by sheepsfoot and rubber-tired rollers in which the load comes into contact with the soil with little or no impact. The pressure increases with time to a maximum, and the rotation of the roller drum or tire causes a small kneading or shoving as the roller adjusts to the soil surface (Ref 47). These studies led to the development of the kneading and gyratory-shear-compaction methods.

McLeod (Ref 49) has found that the following factors can have a great influence on the efficiency and effectiveness of compacting asphalt concrete to a specified density by the rolling operation:

(1) viscosity-temperature characteristic of the asphalt cement,
(2) temperature of the mix,
(3) rate of increase in density and stability of the mix as rolling proceeds,
(4) rate of cooling of the mix behind the spreader,
(5) type of rolling equipment, and
(6) viscosity of asphalt cement.

To a lesser extent the type of aggregate, gradation of aggregate, asphalt content, and amount of mineral filler can also have an effect on the compaction process.

EFFECT OF TEMPERATURE

The effect of temperature must be considered from two viewpoints. The first is the effect of the temperature on the components of the mixture. A change in the temperature also causes changes in the viscosity and wetting energy of the asphalt (Ref 52). A second effect is the change in the mixing, laying, and compaction operations produced by the alterations in the properties of the mix components.

Griffith (Ref 51) has postulated that there is an optimum viscosity for mixing asphaltic materials containing a given type and gradation of aggregate. The optimum would probably vary from one type and gradation of aggregate to another and is that viscosity at which all the aggregate particles are easily and uniformly coated with asphalt in a given time period. The viscosity could be obtained at a temperature which allows the asphalt to remain in place on the aggregate particles while resulting in a minimum permanent hardening of the asphalt cement.

The temperature of mixing has a great influence on asphalt absorption (Ref 52). As the temperature of mixing is increased the viscosity of the asphalt is decreased allowing a greater amount to be absorbed by the aggregate. Lean mixes may result with the use of a high mixing temperature while a low mixing temperature may give an overly rich appearance because of the variation in the amount of asphalt absorbed.

McLeod (Ref 49) has demonstrated the importance of the temperature of the mix at the time of compaction. He showed that for a given compactive effort the value for both the density and Marshall stability decreases as the temperature of compaction is decreased. McLeod also found that for the same compactive effort the mixes containing the lower viscosity asphalt cement can be compacted at a lower temperature than those with higher viscosity values.

After spreading and compaction of the mix the asphalt cement is subjected to a wide variety of temperatures ranging from subzero to 140° F or greater. In such conditions the response of the asphalt binder to induced temperature gradients (Ref 53) may control performance of the road surface. Adams (Ref 54) has found in experiments with one type of mixture that a definite relationship exists between pavement temperature and the percent compaction or densification under traffic.

It is also known that asphalt-treated mixtures undergo oxidation or hardening beginning immediately after mixing and continuing throughout the pavement life. The major portion of the process occurs very rapidly after the asphalt is brought into contact with the surface of the heated aggregate (Ref 55). After compaction the hardening process continues and is influenced in part by the pavement temperature and the amount of light present. Hveem (Ref 55) has also found, however, that the rate of hardening after mixing under atmospheric conditions is virtually unaffected by the amount of hardening developed during the mixing cycle.

Asphalts can be classified as thermoplastic materials which soften when heated and become more viscous when cooled (Ref 56). Since asphalt is temperature-susceptible, it is apparent that an asphalt-treated specimen which has asphalt as a binder would portray the same characteristics. During the summer months when the mixture absorbs enough heat to soften the asphalt, its load-carrying capacity is drastically reduced (Ref 57). Neppe (Ref 58) advises that the effect of the elevated temperatures is to reduce asphalt viscosities and to reduce the stability of the mixtures by an increase in the lubricating propensities of the asphalt.

At low temperatures the asphalt becomes quite viscous and brittle and reacts like an elastic material. The increased strength observed in an asphalt-treated mixture at low temperatures results from the fact that a decrease in temperature increases the cohesion of the mixture (Ref 59). It has been found from impact tests that as the temperature is lowered, the asphalt-treated mixture can absorb less energy before failure. Neppe (Ref 58) has found that service performance at low and high temperatures is far more dependent on the characteristics of asphalt than at normal temperatures.

Pell (Ref 60) has investigated the fatigue characteristics of asphalt specimens tested in rotating bending under constant stress amplitude and also with torsional oscillations under constant strain. From the test results he postulated that the applied tensile strain is the factor which controls the fatigue life of both the asphalt and asphalt-treated mixtures. Monismith (Ref 32) has found in his controlled-stress repeated load tests that a decrease in temperature results in an increase in fatigue life through its influence on specimen stiffness.

The investigations of the indirect tensile test by Hudson and Kennedy (Ref 7), Messina (Ref 14), and Breen and Stephens (Ref 61) have also shown increased stiffness with decreased temperature. The asphalt concrete becomes more brittle as the temperature is decreased. Although the load to fracture increases slowly with decreasing temperature, the ultimate deflection decreases and the work required to fracture the specimen decreases.

EFFECT OF RATE OF LOADING

Asphalts are considered to be a viscoelastic material, i.e., a material whose stress-strain characteristics are time dependent. For relatively slow loading rates asphalt generally behaves as a viscous material and will flow under the load. On the other hand, as the loading rate is increased asphalt will become more elastic in nature (Ref 62). In fact the asphalt is almost completely elastic at the very rapid loading rates (Ref 25).

A static load produces the most severe loading condition on viscous materials, since low rates of loading are much more critical than high loading rates (Ref 63). The strength as well as the effective modulus of elasticity of an asphalt-treated mixture increases as the loading rate increases (Ref 64).

In an unconfined compression test on asphalt mixtures under constant rates of strain and with increasing stress, Abdel-Hady and Herrin (Ref 65) found that a change of loading rate from 0.005 in/min to 0.08 in/min has no appreciable effect on the maximum strength of a soil-asphalt specimen, while loading rates greater than 1.0 in/min cause a rapid increase in the maximum specimen strength. Their results also indicate that increases in loading rates from 0.08 in/min to 1.0 in/min produce smaller increases in strength values than rates greater than 1.0 in/min.

On the other hand, work (Ref 7) with the indirect tensile test at The University of Texas on hot-mix asphalt samples has shown that for load rates between 0.05 in/min and 0.5 in/min the strength of the specimens increases relatively fast especially at the lower temperatures. For loading rates between 0.5 in/min and 6.0 in/min the specimen strength increases to a lesser extent than for the lower rates. They also found that for temperatures higher than room temperature there is no appreciable difference between the strength values for loading rates greater than 0.5 in/min.

EFFECT OF REPEATED LOADING

The repeated loading effect is important in flexible pavement design because of the numerous repetitions of load the pavement must withstand without failure during its life. Repeated loading effects have not been well documented, primarily because of difficulty in developing standard test procedures.

Fatigue tests (Refs 58, 66, and 67) on asphalt beams have shown that the number of repetitions a beam of asphalt concrete can withstand without fracture increases as the applied load is decreased. Pell (Ref 40) and Monismith, et al (Ref 68) have reported that fatigue life increases with stiffness of the mix. Monismith (Ref 69) has shown that tensile cracking occurs during repeated loading of asphalt paving mixtures.

Successive loadings tend to cause work hardening and result in an increase in the time required for any one load application to bring about a certain deformation (Ref 70). An increased temperature, however, reduces the time necessary for a given load to bring about a certain deformation.

CHAPTER 3. DISCUSSION OF INDIRECT TENSILE TEST AND TEST EQUIPMENT

The increased interest in the tensile properties of asphalt-treated materials has caused investigators to critically evaluate the available tensile tests for theory and validity. There are three tests presently being used for the evaluation of the tensile characteristics of highway materials. They are classified as (1) direct tensile tests, (2) bending tests, or (3) indirect tensile tests.

Hudson and Kennedy (Ref 7) in their evaluation of the three test types concluded that the indirect tensile test presently has the greatest potential for the assessment of the tensile properties of highway materials. On the basis of their evaluation the indirect tensile test will be used to judge the factors affecting the tensile properties of asphalt-treated materials.

Since the theoretical analysis of the indirect tensile test assumes that the circular specimen is an ideal elastic medium with identical tensile and compressive properties, the asphalt-stabilized mixtures evaluated in this study are considered to act as linear elastic materials.

The test involves the loading of a circular element with compressive loads acting along two opposite generators (Fig 1). This loading condition produces a relatively uniform tensile stress distribution perpendicular to and along a portion of the diametrical plane containing the applied load. When the applied tensile stress exceeds the tensile strength of the material, failure usually occurs by splitting along the loaded plane.

THEORY OF THE TEST

The stress analysis of a circular element subjected to loading at its boundary has been investigated by various individuals. Among those who have analyzed the stress distribution are Timoshenko (Ref 21), Frocht (Ref 17), Muskhelishvili (Ref 71), Sokolnikoff (Ref 72), and Wright (Ref 73).

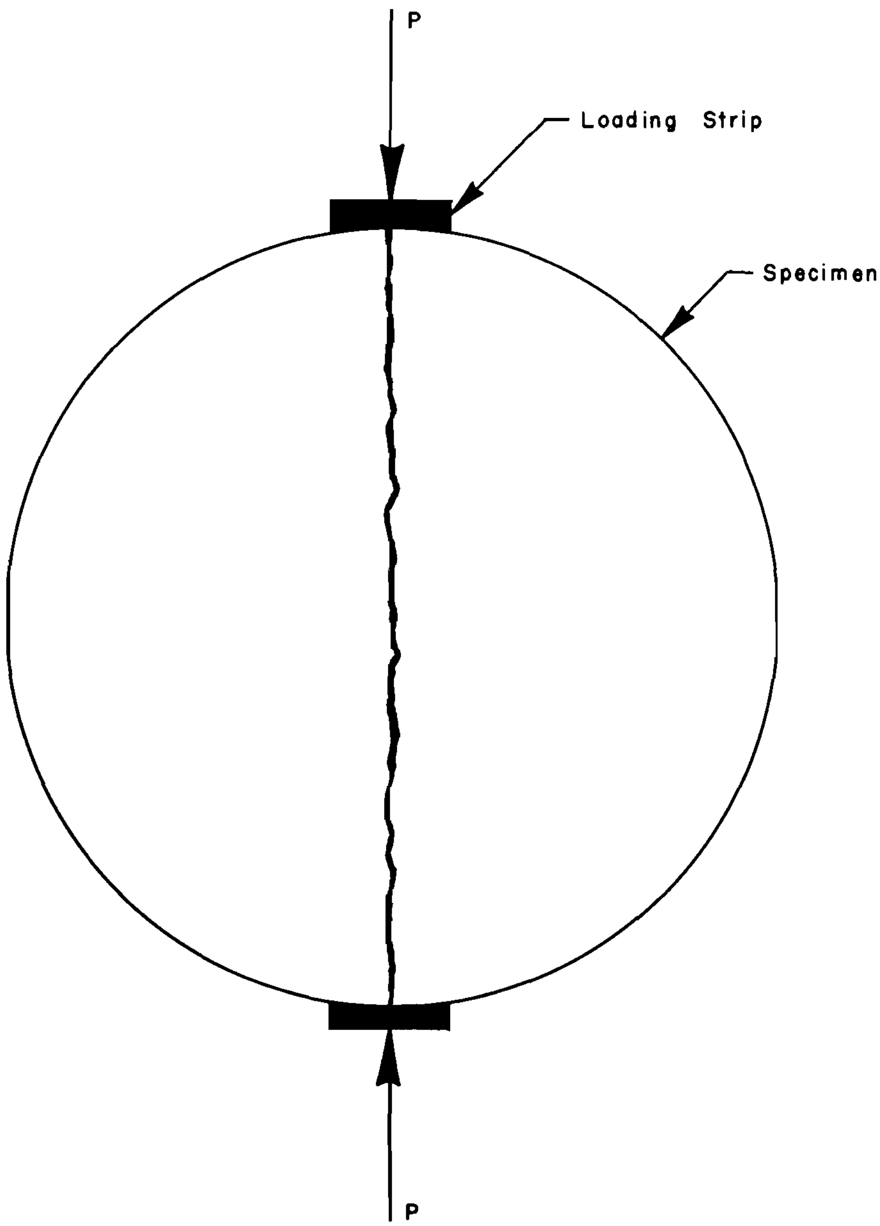

Fig 1. The indirect tensile test.

Hondros (Ref 23) has analyzed the circular element supporting a short strip loading (Fig 2) assuming that the body forces (self-weight) are negligible. In this case the stress distribution for plane stress (disc) and plane strain (cylinder) are identical. Hondros' equations for the stresses along the principal diameters are presented below:

(1) Stresses along the Vertical Diameter (OY)

$$\sigma_{\theta y} = +\frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\sin 2\alpha}{(1 - 2r^2/R^2\cos 2\alpha + r^4/R^4)} - \tan^{-1}\left(\frac{(1 + r^2/R^2)}{(1 - r^2/R^2)}\tan\alpha\right)\right] \quad (1)$$

$$\sigma_{ry} = -\frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\sin 2\alpha}{(1 - 2r^2/R^2\cos 2\alpha + r^4/R^4)} + \tan^{-1}\left(\frac{(1 + r^2/R^2)}{(1 - r^2/R^2)}\tan\alpha\right)\right] \quad (2)$$

$$\tau_{r\theta} = 0$$

(2) Stresses along the Horizontal Diameter (OX)

$$\sigma_{\theta x} = -\frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\sin 2\alpha}{(1 + 2r^2/R^2\cos 2\alpha + r^4/R^4)} + \tan^{-1}\left(\frac{(1 - r^2/R^2)}{(1 + r^2/R^2)}\tan\alpha\right)\right] \quad (3)$$

$$\sigma_{rx} = +\frac{2p}{\pi}\left[\frac{(1 - r^2/R^2)\sin 2\alpha}{(1 + 2r^2/R^2\cos 2\alpha + r^4/R^4)}\right.$$

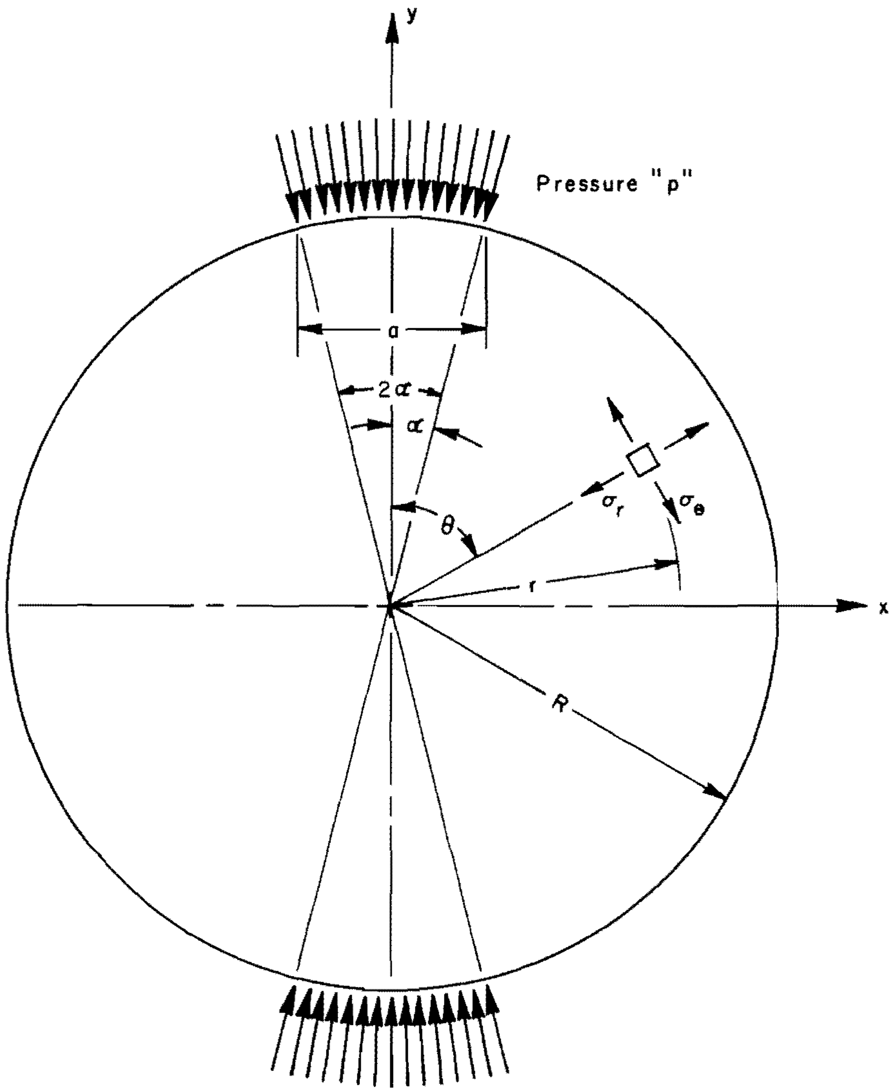

Fig 2. Notation for polar stress components in a circular element compressed by short strip loadings (from Ref 23).

$$- \tan^{-1} \left(\frac{(1 - r^2/R^2)}{(1 + r^2/R^2)} \tan \alpha \right) \Bigg] \tag{4}$$

$$\tau_{r\theta} = 0$$

The stresses along these principal planes corresponding to the diameters through the OX and OY axes, for a loading strip width less than D/10, are plotted in Fig 3. For this case the equations for the stresses at the center would reduce to:

$$(1) \quad \sigma_{\theta y} = \sigma_{rx} = + \frac{2p\alpha}{\pi} = + \frac{2P}{\pi Dt} \tag{5}$$

and

$$(2) \quad \sigma_{\theta x} = \sigma_{ry} = - \frac{\sigma p\alpha}{\pi} = - \frac{6P}{\pi Dt} \tag{6}$$

where

$r/R = 0$,

$P = pat$,

$\alpha = a/D$, and

$p\alpha = P/Dt$.

These expressions agree with Wright's development (Ref 73). This illustrates that point loads and short distributed loads applied to circular elements develop identical stresses at the center of the specimen.

STANDARD TEST PROCEDURES

The test procedures utilized in this study were essentially the same as recommended by Hudson and Kennedy (Ref 7) with all specimens tested at room temperature (77^{o} F) at a loading rate of 2.0 inches per minute. The major

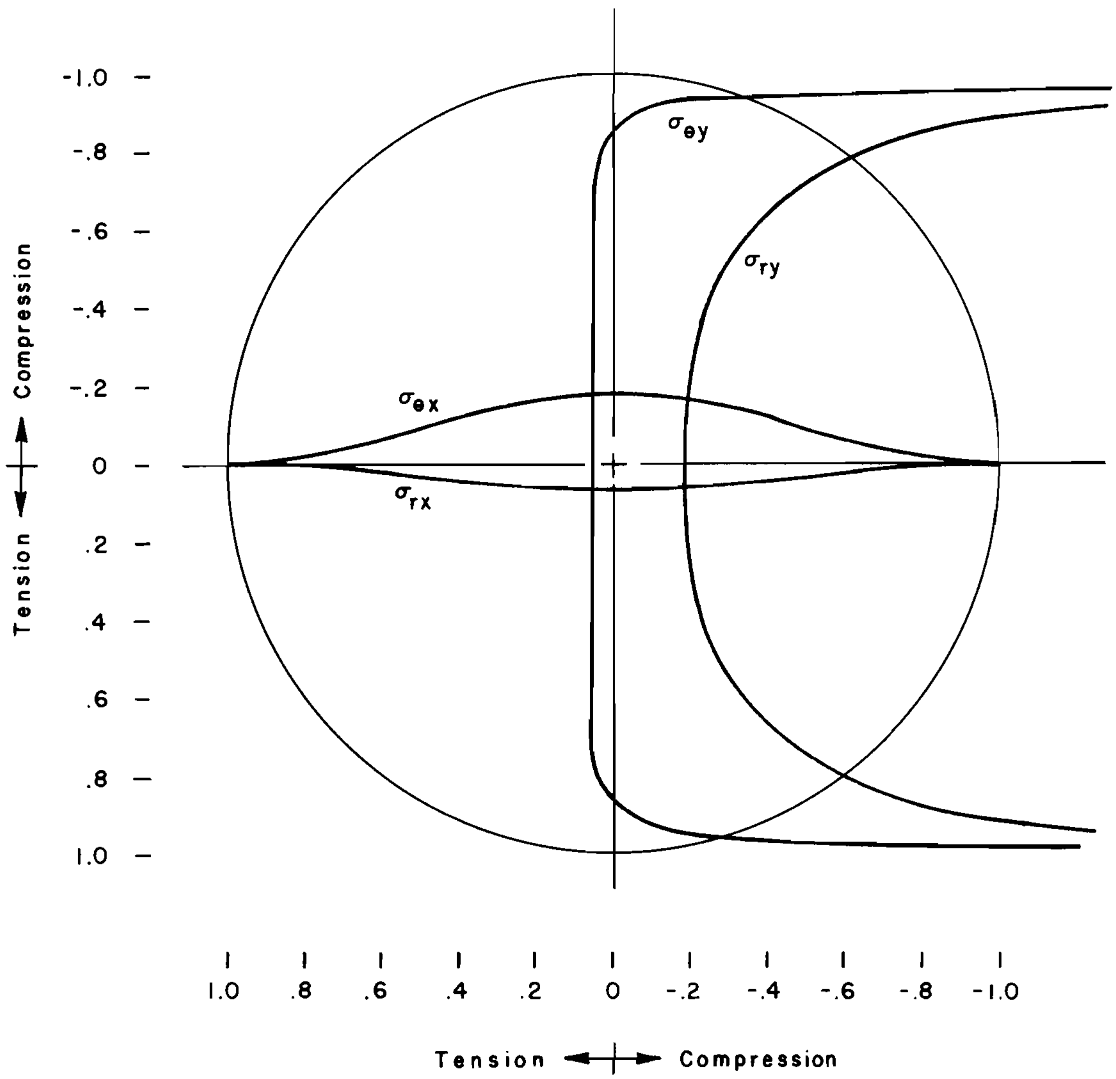

Fig 3. Stress distribution along the principal axes for loading strip width (a) less than D/10. $2p/\pi = 1$

exception to the procedures originally proposed by Hudson and Kennedy was the type of loading strip used during actual specimen testing. It had been tentatively recommended that future testing utilize a flat loading strip composed of stainless steel 1.0 inch wide. However, utilizing Hondros' development (Ref 23), consideration was given to the configuration of the loading strip, i.e., curved versus flat. A decision was made to use a curved strip instead of a flat one in the test procedure to evaluate the tensile properties of asphalt-treated materials. This decision was based upon the following information:

(1) Utilizing a curved strip with a radius of approximately the same dimension as the specimen radius, the original specimen configuration can be more closely maintained throughout the test. The use of a flat loading strip induces crushing in the periphery of the test specimen, thereby changing its configuration. Any major change in the shape of the specimen can change the stress distribution.

(2) The theoretical development used in this study to formulate the equations for the tensile properties of stabilized materials is based upon the biaxial loading of a circular element at its boundary with a short loading strip. In order to obtain a correct equation for the tensile strength of a linear-elastic material, it is imperative that the area of the specimen loaded during the test be known. It is not possible to calculate or even estimate closely the loading area experienced by an asphalt-treated specimen during testing with a flat loading strip. In fact, the loading area will vary depending upon the modulus of the material as well as the magnitude of the load applied. As the specimen is initially contacted by the flat strip, a line of infinite stress immediately forms at the specimen surface causing the specimen to be crushed. As the loading head continues to move downward, the specimen is continually crushed until a loaded area sufficient to withstand the compressive force is formed. The loading area for a flat loading strip, therefore, changes from a line to a rectangular area. It is not possible, however, to predict what the dimensions of the final loading area will be. On the other hand, the curved strip results in a known loading area and allows the use of the theoretical equation required for evaluating linear elastic materials.

The dimensions and configuration of the loading strip used in the tests described herein are shown in Fig 4. The overall width of the strip is one inch with the middle 1/2 inch of the strip composed of a curved section with a radius of 2 inches. Tangent sections approximately 1/4-inch long were then machined from the curved portion to each end of the strip. The edges of

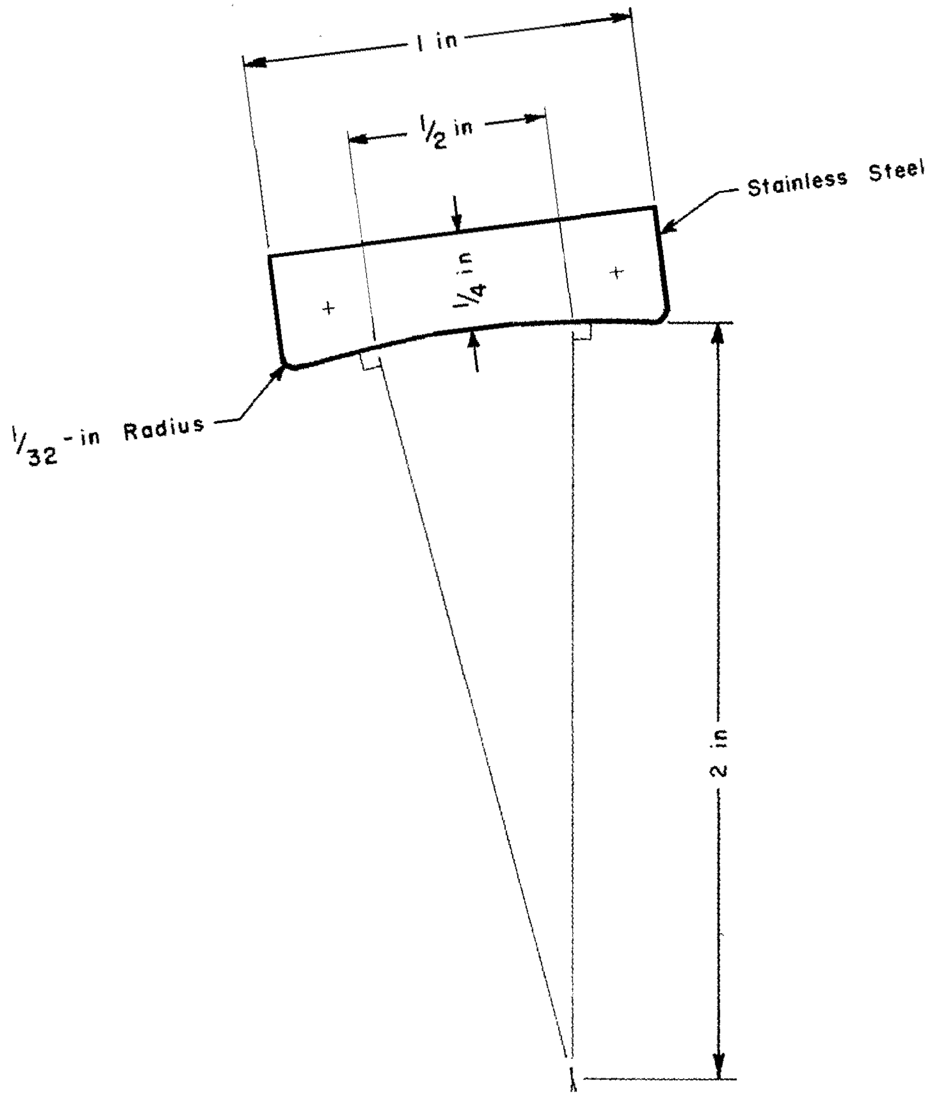

Fig 4. Curved loading strip configuration.

the strip were rounded to a radius of approximately 1/32 inch. The tangent sections and rounded edges were used to prevent any punching of the specimen by the sharp edge of the loading strip during testing. With this configuration it is expected that some seating of the strip with the specimen will occur during the initial loading stages of the indirect tensile test of the materials evaluated in this study.

The stresses along the principal planes corresponding to the horizontal and vertical axes for a loading strip width of one inch are plotted in Fig 5. The equations for the stresses at the center of a 4-inch-diameter specimen for this loading configuration reduce to

$$(1) \quad \sigma_{\theta y} = \sigma_{rx} = 0.46288 \frac{P}{\pi t} = 0.14734 \frac{P}{t} \tag{7}$$

$$(2) \quad \sigma_{\theta x} = \sigma_{ry} = -1.47360 \frac{P}{\pi t} = -0.46906 \frac{P}{t} \tag{8}$$

TEST EQUIPMENT

The basic testing equipment is shown in Fig 6 and consists of an adjustable loading frame, a closed-loop electrohydraulic loading system, and a loading head. The loading frame is a modified, commercially available shoedie with upper and lower platens constrained to remain parallel during testing (Fig 7). The vertical deformation of the specimen is measured by a DC linear variable-differential transducer which also is used to control the rate of load application by providing an electrical signal related to the relative movements of the upper and lower platens. The measurements are recorded on an X-Y plotter.

Horizontal deformations of the test specimen are obtained through the use of a measuring device consisting of two cantilevered arms with strain gages attached, as shown in Fig 8. Movements or deflections of the arms at the point of contact with the specimen have been calibrated with the output from the strain gages. The horizontal measurements are recorded on an X-Y plotter.

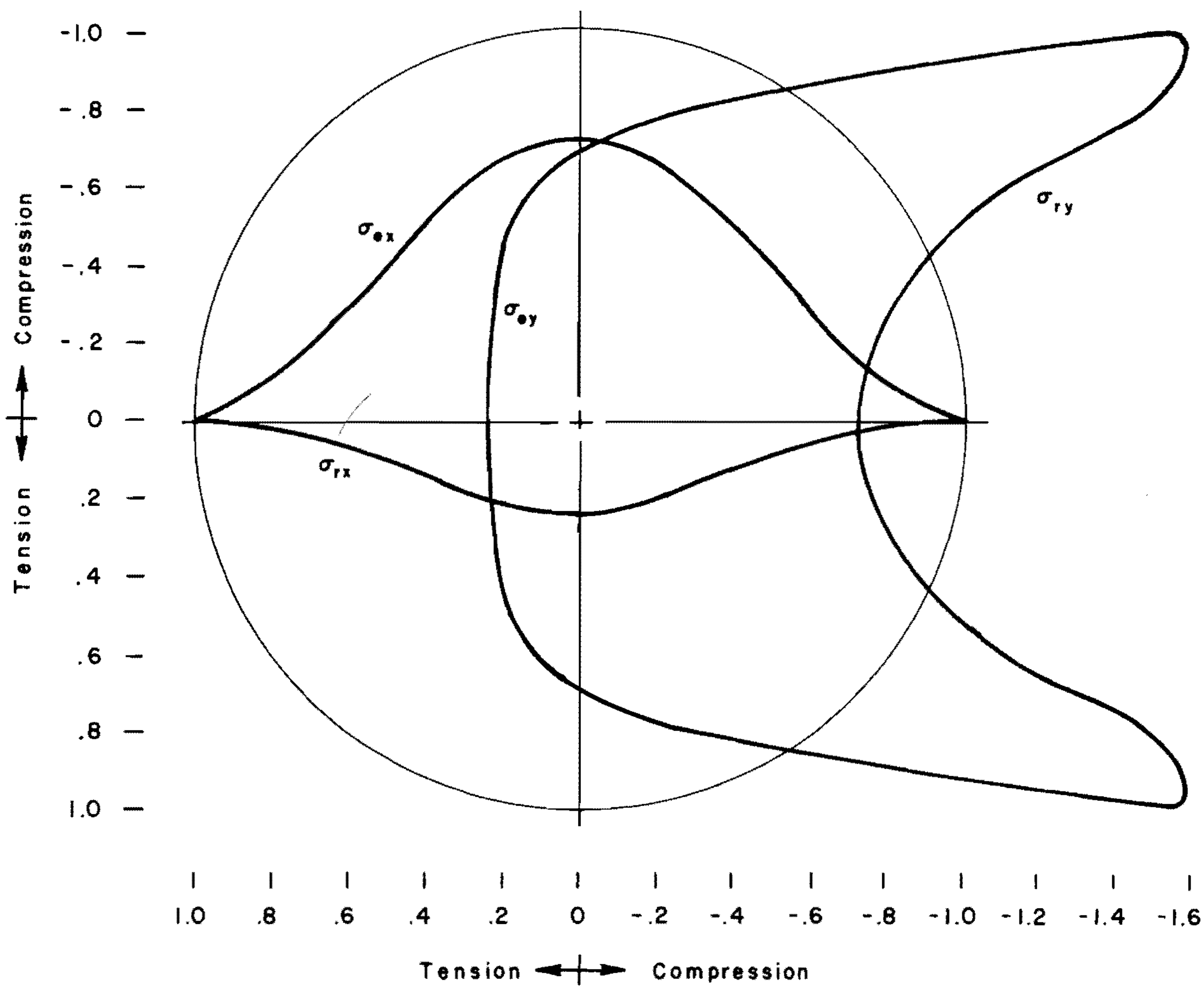

Fig 5. Stress distribution along the principal axes for a loading strip width equal to one inch. $2p/\pi = 1$

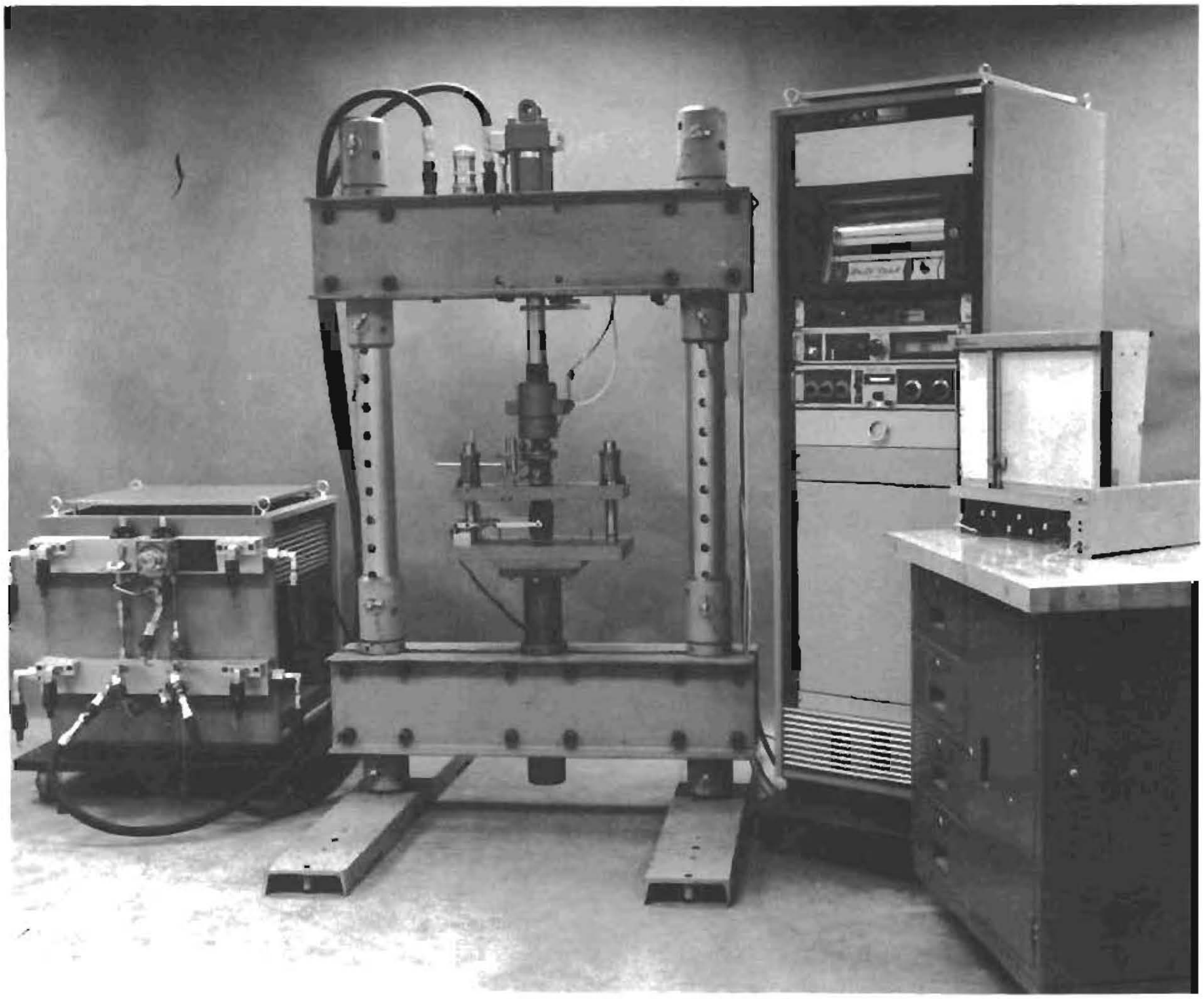

Fig 6. Basic indirect tensile testing equipment.

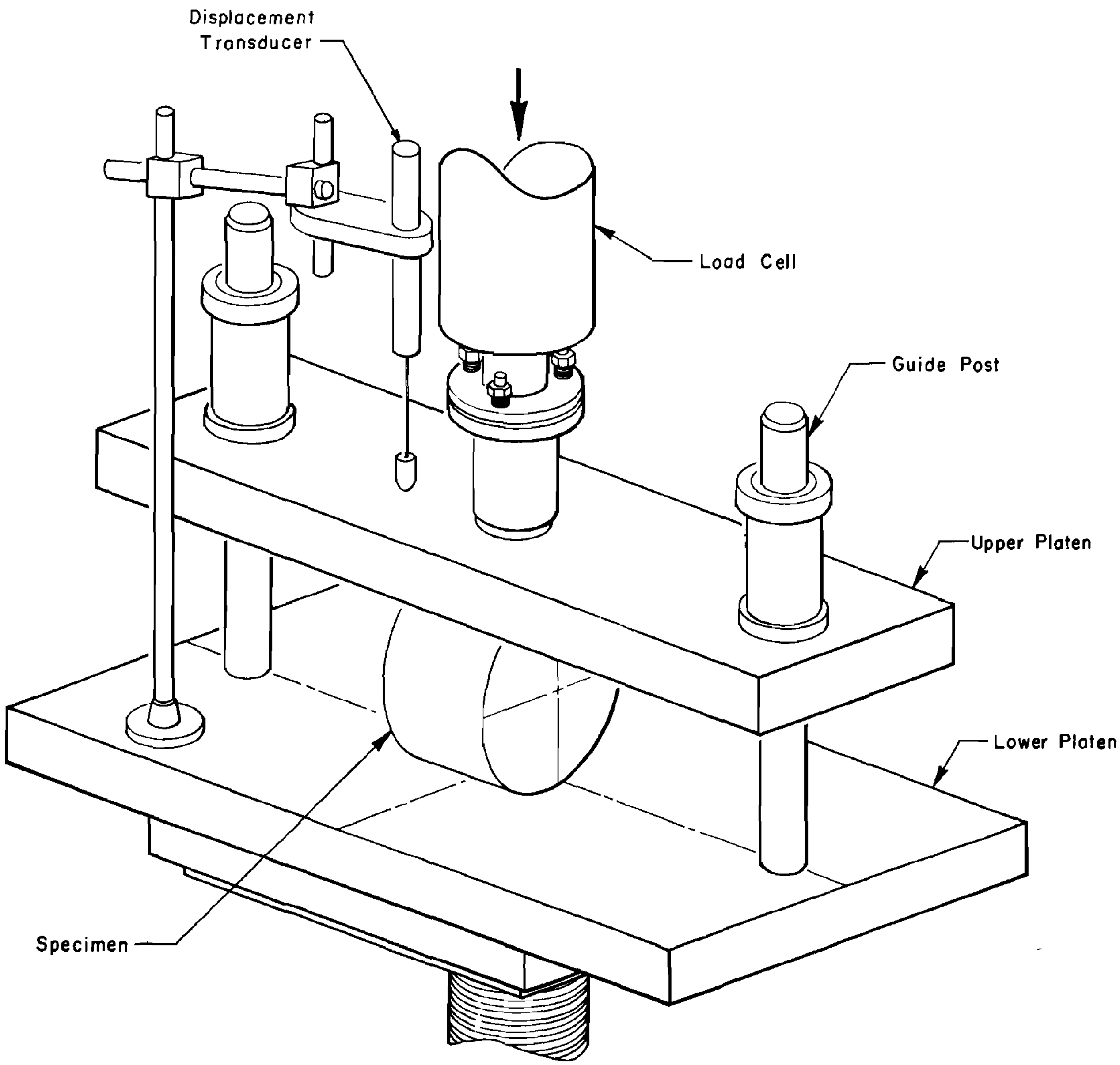

Fig 7. Loading head with rigid parallel platens.

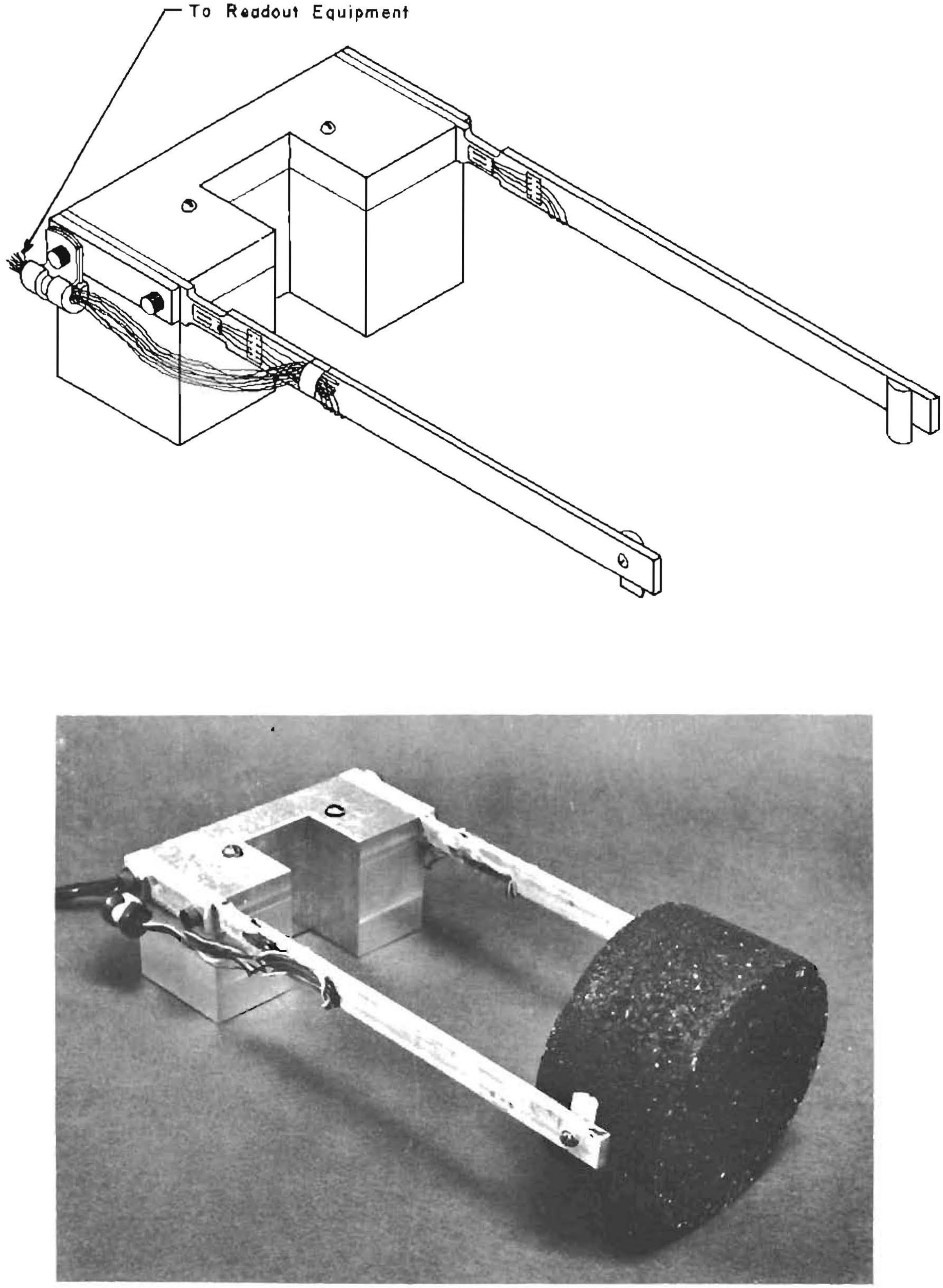

Fig 8. Lateral-strain measuring device.

CHAPTER 4. EXPERIMENTAL PROGRAM

This program was designed to investigate the significance of main effects, two-way interactions, and selected three-way interactions for two levels of eight different factors considered to affect the tensile properties of asphalt-treated materials. The factors and levels selected for this investigation are summarized in Table 3 and discussed in subsequent paragraphs.

TABLE 3. FACTORS AND LEVELS SELECTED FOR EXPERIMENTAL PROGRAM

Factor	Level	
	Low	High
Aggregate type	Crushed limestone	Seguin gravel (rounded)
Aggregate gradation	Fine	Coarse
Asphalt viscosity (specification)	AC-5	AC-20
Asphalt content	3.5%	7.0%
Compaction type	Impact	Gyratory-shear
Mixing temperature	250° F	350° F
Compaction temperature	200° F	300° F
Curing temperature	40° F	110° F

SELECTION OF FACTORS

Aggregate Type

As discussed in Chapter 2 the aggregate type and aggregate characteristics have a great effect on the strengths of asphalt-treated mixtures. To investigate relatively extreme aggregate types, Seguin gravel and crushed limestone were selected for the levels of the first factor. The Seguin gravel consists of a naturally occurring subrounded nonporous particle with a relatively smooth surface texture. The limestone, on the other hand, is a naturally occurring porous aggregate which when crushed forms angular particles with relatively rough surface texture.

Aggregate Gradation

The type of gradation which should be used for stabilized materials is a controversial subject (Ref 37). To obtain the widest reasonable range possible for the levels of this factor, fine and coarse gradations were selected based upon Winterkorn's classification of soils for asphalt stabilization (Ref 74) and Texas Highway Department Specification Item 346 (Ref 75). These gradations generally fall within Winterkorn's classification Types A and C as outlined in Table 4 and as shown in Figs 9 and 10.

TABLE 4. WINTERKORN'S CLASSIFICATION OF SOILS FOR ASPHALT STABILIZATION (from Ref 74)

Sieve Analysis Passing	Sand-Gravel Asphalt %		
	A	B	C
1-1/2"	100		
1"	85-100	100	
3/4"	65-85	80-100	100
No. 4	40-65	50-75	80-100
No. 10	25-50	40-60	60-80
No. 40	15-30	20-35	30-50
No. 100	10-20	13-23	20-35
No. 200	8-12	10-15	13-30

The fine gradation corresponds very closely to the Texas Highway Department Specification Type D fine graded surface course material (Ref 75). The coarse gradation also closely corresponds to the Texas Highway Department

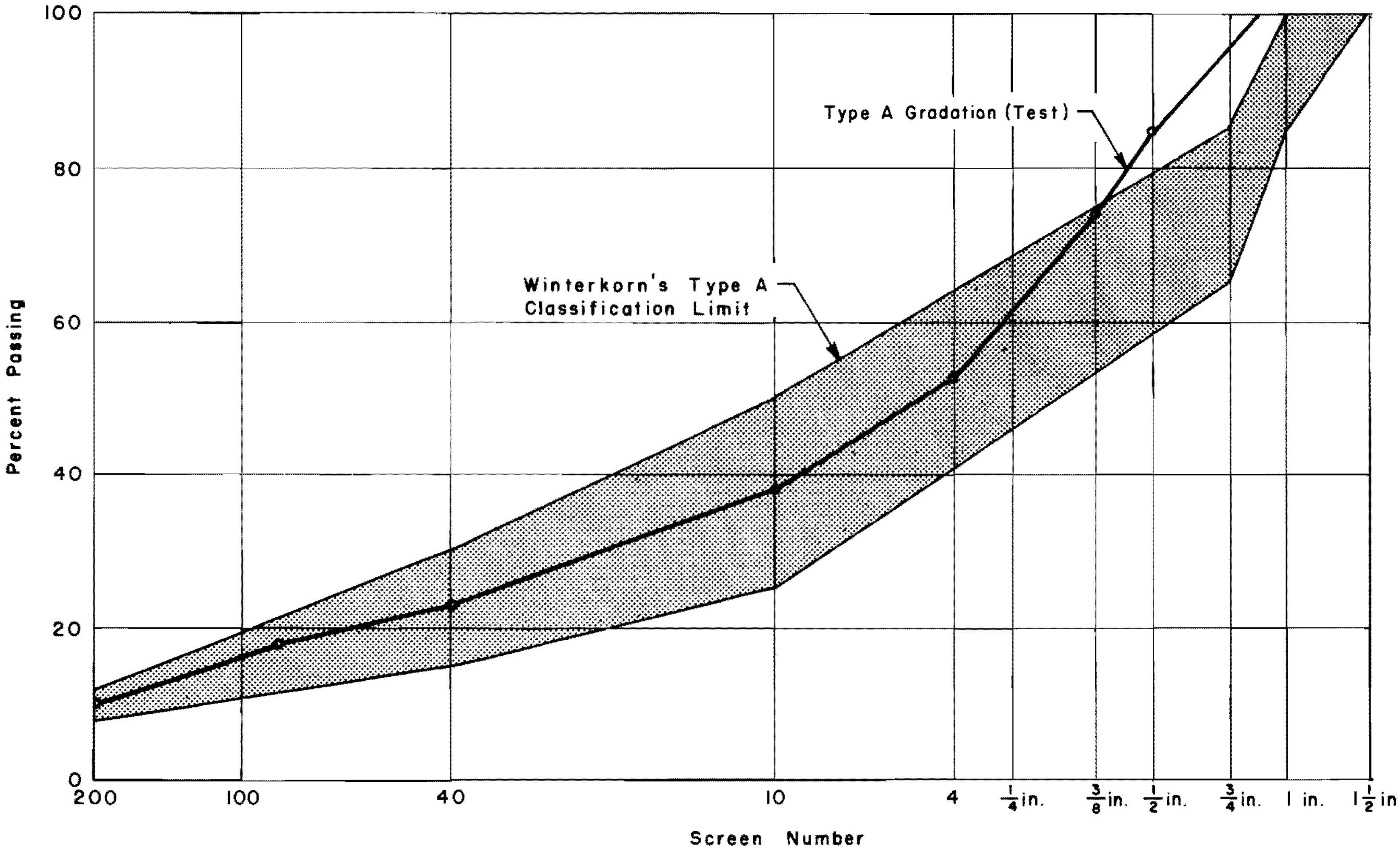

Fig 9. Comparison of test gradation A with Winterkorn's classification Type A (coarse gradation).

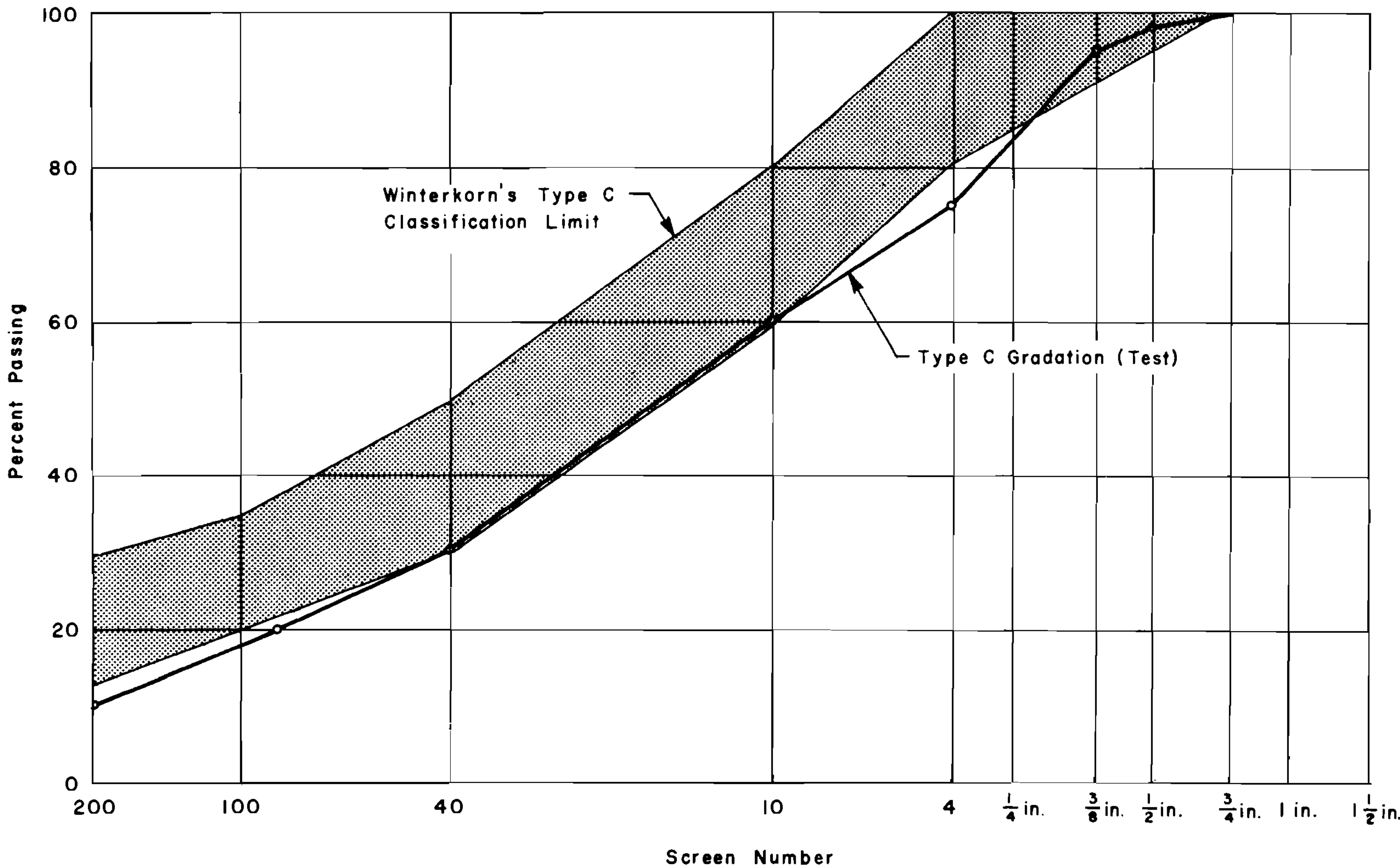

Fig 10. Comparison of test gradation C with Winterkorn's classification Type C (fine gradation).

Specification Type A coarse graded base course material (Ref 75). Aggregate larger than 7/8 inch was not included in the selected gradations because the size of the test specimen (2 inches thick by 4 inches in diameter) limits the maximum size of aggregate (Ref 80). Comparisons of the selected gradations and Texas Highway Department Specification Types A and D are presented in Appendix 1.

ASPHALT VISCOSITY

The effects of asphalt viscosity are important in mixing, spreading, and compacting the mixtures on the road (Ref 76).

Rader (Ref 77) has stated that in certain types of mixtures, stability is affected by the consistency of the asphalt binder, with the binders of higher consistency producing mixtures of greater stability.

To investigate Rader's postulation it was decided to vary the viscosity of the asphalt-treated material used. The types of asphalts selected were AC-5 and AC-20. The viscosity-temperature relationship of each type is shown in Fig 11. The test data on both asphalt cements are presented in Table 5.

The asphalt cements selected are readily available and are widely used in hot mix asphalt mixtures.

ASPHALT CONTENT

Experience has proven that the use of the highest asphalt content possible in a paving mixture consistent with adequate stability for the anticipated loading conditions is a good design principle (Ref 78). The range of asphalt contents usually used to stabilize soils and other materials extends from 4.0 to 10.0 percent of total mixture weight. The limits of the range of asphalt content used in this study were obtained from the Texas Highway Department specifications (Ref 75). The lower limit of asphalt contents recommended for Texas Highway Department Specification Types A and D is 3.5 percent while the upper limit is 7.0 percent. These two were selected as the levels to be investigated in this experiment.

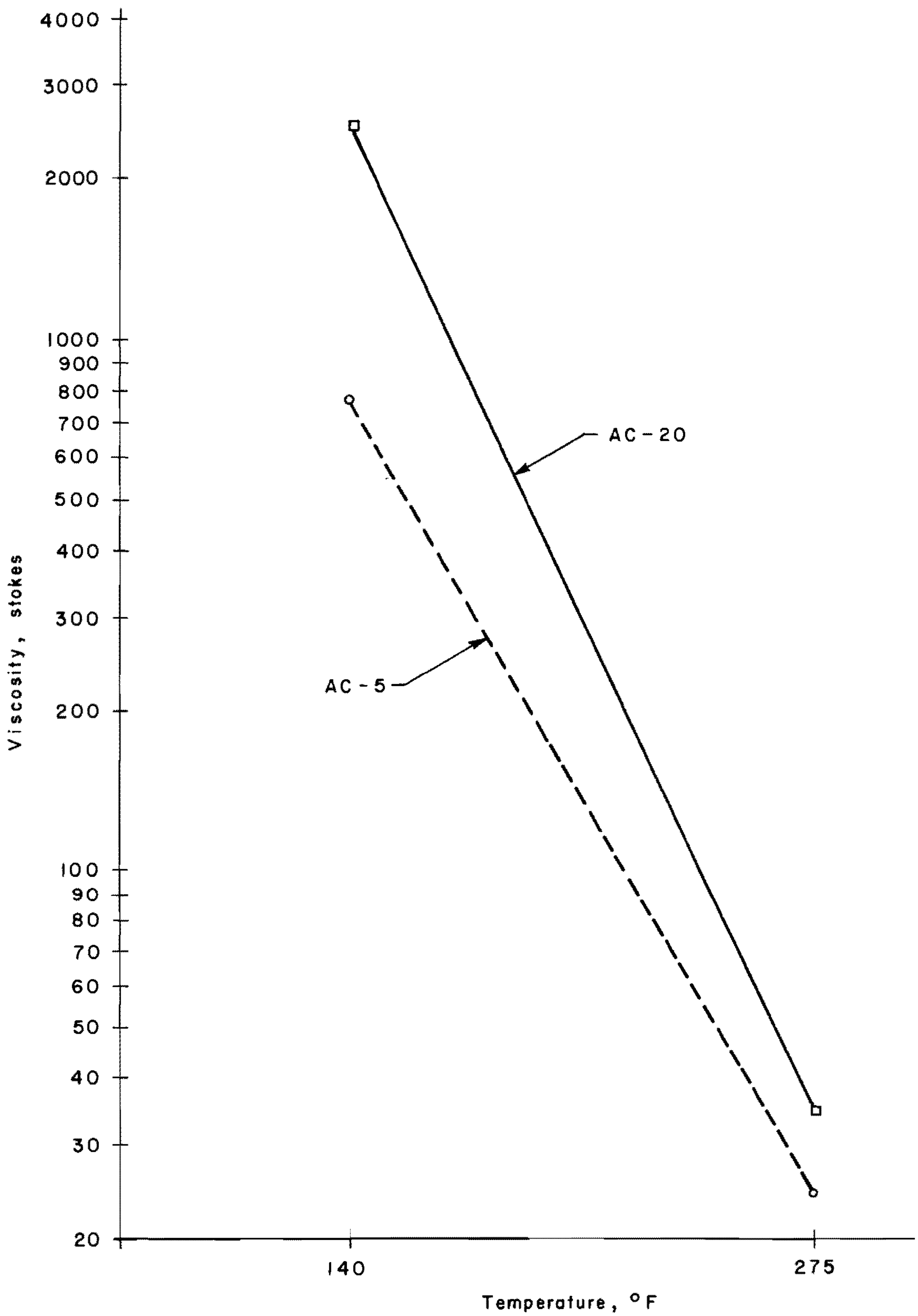

Fig 11. Temperature-viscosity relationship of asphalt cements used in the experiment.

TABLE 5. TEST DATA FOR COSDEN AC-5 AND AC-20 ASPHALT CEMENTS (Source: Cosden Petroleum Corporation, Big Springs, Texas)

Asphalt	AC-5	AC-20
Water, %	NIL	NIL
Viscosity at 275° F, stokes	2.45	3.6
Viscosity at 140° F, stokes	773	2532
Flash point C.O.C., ° F	560	565
Ductility, 77° F, 5 cm/min, cm	141+	141+
Relative viscosity (after oxidation, 15 μ films for 2 hours at 225° F, viscosities determined at 77° F)	3.87	2.7
Penetration at 77° F, 100 g, 5 sec	112	64
Specific gravity at 77° F	1.003	1.009
Solubility in CCl_4, %	99.7+	99.7+

COMPACTION TYPE

McRae (Ref 46) presents the view that stabilities obtained from specimens compacted by impact are higher than the stabilities on actual pavement cores of equivalent density and asphalt content. He further postulates that this indicates a difference in structure or aggregate particle arrangement and distribution. On the other hand, Lefebre (Ref 79) has accumulated considerable evidence to show that for many pavements 100 percent of laboratory compacted density by the 75-blow Marshall hand compactor approximates the ultimate density a pavement achieves in service under heavy traffic loads.

According to Nevitt (Ref 80) the static compression method of compaction has three major deficiencies. The first is that a very high force intensity is required to produce densification of the mixture. Secondly, with many aggregates the method produces excessive degradation of particles. The method also induces disproportionately large side wall effects. The deficiencies outlined above were felt to be sufficient justification for the elimination of this method from consideration in this investigation.

Seed, et al (Ref 81) have investigated the effect of the impact and kneading compaction on the stability of silty and sandy clay soils tested in the Hveem stabilometer. The relationship of density versus stability for the two methods was similar for the soils. The kneading compactor, however, produced slightly higher stabilities at the lower densities while the impact resulted in somewhat higher stabilities at the higher densities.

Specimens compacted with the gyratory-shear compactor appear to exhibit stability values which more closely agree with field cores than do those compacted by impact (Ref 46). McRae believed that this indicated that the structure or particle arrangement of the specimen compacted with the gyratory-shear compactor simulated more closely the prototype conditions than of those specimens compacted by impact.

The impact-compaction method was selected as the lower limit of compaction type because it does not actually simulate field rolling closely and does not necessarily produce a good representation of field samples. There is evidence that similar results for Hveem stabilities can be obtained with both the impact and kneading-compaction methods; however, the kneading method was thought to represent a better simulation of field rolling and was considered to be at a middle level between the impact and gyratory-shear methods. Because the gyratory-shear-compaction method appears to produce the best representation of field densities, it was selected as the upper level of the compaction type under consideration in this study.

The impact-compaction procedure used is the standard procedure outlined in Section 3.5 of ASTM Designation D 1559-65, "Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus" (Ref 82). Seventy-five blows were applied to each face during actual specimen compaction. There was one deviation from the ASTM standard procedure. The maximum size of aggregate used in the asphalt-treated mixes was 7/8 inch in diameter. This limitation was due to the maximum size aggregate allowed in the gyratory-shear-compaction method.

The standard gyratory-shear-compaction procedure utilized in this study is the one outlined in the Texas Highway Department Test Method Tex-206-F, Part II (September 1966).

MIXING TEMPERATURE

The viscosity as well as the temperature susceptibility of asphalt cements varies greatly from one type and source to another. Because of the wide variation in these properties an evaluation of mixing temperature to determine the optimum viscosity would have to be based upon a range of temperature values. Griffith (Ref 51) using this type approach has evaluated the mixing temperature-viscosity relationship and has suggested temperature ranges for use in mixing asphalt-treated materials. His recommendations are outlined in Table 6.

TABLE 6. SUGGESTED TEMPERATURES FOR USE IN MIXING ASPHALT-TREATED MATERIALS (from Ref 51)

Penetration Value of Asphalt Cement	Suggested Mixing Temperatures
40- 50	300°-350° F
60- 70	275°-325° F
85-100	275°-325° F
120-150	275°-325° F
200-300	200°-275° F

The asphalt types used in this study are an AC-5 and AC-20 with penetration values of 112 and 64, respectively. The overall temperature range for mixing these asphalt cements as recommended by Griffith is 275 to 325° F. The Asphalt Institute (Ref 45) recommends that in no instance should the asphalt cement be heated to a temperature exceeding 350° F.

Based upon the Asphalt Institute's recommendation, the upper level of mixing temperature was set at 350° F. The lower level was set arbitrarily at 250° F. These temperatures are both outside Griffith's recommended range by 25° F. These selections should provide a good estimate of the effect of relatively extreme temperature differences on the tensile properties of asphalt-treated materials.

COMPACTION TEMPERATURE

The temperature of the mix or more specifically the viscosity of the asphalt has a great influence on the compaction of asphalt-treated mixtures. The lowest mix temperature which produces the specified end results in the finished pavement is desired for hot mix asphalt-treated concrete (Ref 83).

Parker (Ref 84) investigated the effects of compaction temperatures ranging from 100 to 350° F on the density of specimens compacted by the Marshall method. The density used as the basis of comparison in the study by Parker was that obtained at a compaction temperature of 275° F. He found that ranges in temperatures from 275 to 350° F had little effect on the density obtained; however, as the temperature drops below 275° F, there is an immediate reduction in density with a rapid loss occurring at a temperature of 225° F. From his study Parker recommended that compaction should be largely completed by the time the temperature reaches 225° F, while the mix is still in a plastic state.

Parker (Ref 85) and Nijboer (Ref 86) have both concluded that rolling below 175° F is not effective in the compaction of asphalt pavement layers. Monismith (Ref 25) recommends a range of temperatures from 200 to 275° F for the initial rolling during the construction of an asphalt layer.

The lower level of the compaction temperature factor was selected as 200° F. This corresponds with the temperature cited by Monismith (Ref 25) and falls within the limits reported by Parker (Refs 84 and 85) and Nijboer (Ref 86). An upper level of 300° F was selected, based upon the recommended temperature of 275° F and the previously selected mixing temperature of 350° F. The selection of 300° F as an upper limit maintained a constant temperature differential of 50° F between the lower and upper temperature levels of the mixing and compaction temperatures.

CURING PROCEDURE

After the compaction process, the asphalt-treated materials are subjected to a wide variety of temperatures, ranging from subzero to about 140° F. The temperature of the pavement has a great effect upon the aging or hardening of the asphalt layer. The major contributors to the hardening process are the rates of oxidation and volatilization (Refs 25, 87, and 88).

Oxidation is the reaction of oxygen with the asphalt component of the stabilized layer. At normal temperatures the rate of oxidation reaction is slow, therefore allowing the oxygen to be absorbed by the asphalt; however, at increased temperatures the rate of oxidation increases, leading to advanced weathering.

Volatilization is the evaporation of the lighter constituents from the asphalt mixture. This process is also accelerated by increased temperatures. The asphalt type, source, and refining process can also have an effect upon the possible rate of volatilization.

In order to consider the curing effects due to the different seasons of the year, the levels of curing temperatures selected were 40° F and 110° F. The two temperature levels were chosen as being good estimates of the average low and high pavement temperatures indicative of the winter and summer months, respectively.

PARAMETERS EVALUATED

In this study the following parameters were evaluated:

(1) Indirect Tensile Strength (see Eq 7, page 23)

$$S_T = \frac{0.14734 \; P_{max}}{t}$$

where

P_{max} = maximum total load, pounds, and

t = average height of specimen, inches.

(2) Horizontal Failure Deformation - Horizontal deformation of the specimen in inches at the maximum load as recorded on the load-horizontal deformation plot.

Consideration was also given to the evaluation of the three additional parameters defined below:

(1) Vertical Failure Deformation - Vertical deformation of the specimen in inches at the maximum load including any deformation in the upper platen and the loading strip. This deformation was assumed to be equal to the movement of the upper platen from the point of initial

load application to the point of maximum load as measured by the DCDT and recorded on the load-vertical deformation plot.

(2) Tangent Modulus of Vertical Deformation - Slope per unit thickness of the load-vertical deformation relationship prior to failure as defined by a regression analysis.

(3) Deflection Ratio - The ratio between the slope per unit thickness of load-horizontal deformation plot and the slope per unit thickness of the load-vertical deformation plot. Both relationships are defined by a regression analysis.

These three, however, were not evaluated, because there was a question concerning the adequacy of the load-vertical deformation curves. During this test series the curved loading strips were taped to the upper and lower platens of the test apparatus. Subsequent indirect tensile tests on circular aluminum specimens showed the importance of correct loading strip alignment and a good fit between the surfaces of the platens and the backs of the loading strips. The use of tape to secure the loading strips to the platens, therefore, did not provide positive control of these two factors from one specimen to another and cast doubt on the validity of using the vertical deformation data in the analysis of the total experiment. The tests on the aluminum specimen also indicated that the loads applied and the horizontal deformations obtained at each level were apparently unaffected by any slight errors associated with the fit or alignment of the loading strips. Based upon the information above, the tensile strength and horizontal deformation values for the specimens in this study are considered to be correct and can be used as indicators of the tensile properties of asphalt-treated materials. Data for the three parameters associated with the vertical deformation, however, are of questionable value and must be discarded. In future studies the alignment and fit of the loading strips will be closely controlled, thereby allowing the five variables discussed above, as well as others, to be used in the evaluation of tensile properties of asphalt stabilized materials.

STATISTICAL DESIGN AND ANALYSIS

Adequate investigation of a large number of factors requires a statistical design which allows the results to be applicable over the ranges of factors considered. A completely randomized factorial experiment provides this capability since all levels of each factor are represented in combination with all levels of every other factor. Such a complete factorial then allows the investigation of all main effects or factors as well as the effects of each factor or combination of factors on another factor or factors. The effect of one or more factors on others has previously been defined as the interaction* between those particular factors. In a full factorial experiment composed of eight independent factors it is possible to evaluate all interactions ranging from two-way interactions up to an eight-factor interaction.

When a complete factorial experiment requires a large number of test specimens, it is often desirable to select only a fraction of the total number of combinations for testing. This type of fractional factorial design can be used, providing that certain assumptions are met. The assumption normally made for fractional factorials is that certain higher order interactions can be considered to be negligible or zero.

The statistical technique used in this experiment (completely randomized design) also assumes that the errors are normally and independently distributed. Situations sometimes occur where this assumption is not met. One of the most common causes of correlation between errors is time trends in the experimental units. To insure that there are no day-to-day time trends, the decision was made to complete the experiment in one day's time.

A complete factorial experiment for the study described herein would have required 256 specimens. This number greatly exceeded the number of specimens which could be prepared in one working day. The decision was made, therefore, to use a 1/4 replicate of the complete factorial. This reduced the required number of specimens to 64, a level which could be completed in one working day.

The fractional factorial is described by the identity $I = ABCDE = DEFGH = ABCFGH$. The treatment combinations for the experiment are outlined in Table 7. A discussion of this type experimental design is presented by Kempthorne (Ref 89).

The analysis of the experimental data assumes that the preparation and testing procedures were completely randomized. A slight departure from the completely randomized design occurred in the mixing-compaction sequence of the preparation phase. The order of mixing was completely randomized while the

*A two-way interaction is a measure of the failure of a factor to produce the same effect for each level of the second factor and vice versa. Correspondingly a three-way interaction is a measure of the failure of a factor to produce the same effect for each combination of the other two factors.

TABLE 7. TREATMENT RESULTS

Spec. No.	Test Order	Level of Factor A	B	C	D	E	F	G	H	Spec. No.	Test Order	Level of Factor A	B	C	D	E	F	G	H
20	5	1	1	1	1	1	1	1	1	54*	49	2	1	1	1	2	2	1	1
21	39	1	1	1	1	1	1	2	2	55	50	2	1	1	1	2	2	2	2
22	59	1	1	1	1	1	2	2	1	56	35	2	1	1	1	2	1	2	1
23	48	1	1	1	1	1	2	1	2	57	6	2	1	1	1	2	1	1	2
24	22	2	2	1	1	1	1	1	1	58	45	1	2	1	1	2	2	1	1
25	64	2	2	1	1	1	1	2	2	59	4	1	2	1	1	2	2	2	2
26	16	2	2	1	1	1	2	2	1	60	33	1	2	1	1	2	1	2	1
27	53	2	2	1	1	1	2	1	2	61	13	1	2	1	1	2	1	1	2
28	42	2	1	2	1	1	1	1	1	62	21	1	1	2	1	2	2	1	1
29	7	2	1	2	1	1	1	2	2	63	40	1	1	2	1	2	2	2	2
30	30	2	1	2	1	1	2	2	1	64	3	1	1	2	1	2	1	2	1
31	62	2	1	2	1	1	2	1	2	90	68	1	1	2	1	2	1	1	2
32	52	1	2	2	1	1	1	1	1	66	12	2	2	2	1	2	2	1	1
89	67	1	2	2	1	1	1	2	2	67	31	2	2	2	1	2	2	2	2
34	19	1	2	2	1	1	2	2	1	68	20	2	2	2	1	2	1	2	1
35	54	1	2	2	1	1	2	1	2	69	66	2	2	2	1	2	1	1	2
36*	9	1	1	1	2	2	1	1	1	70	8	2	1	1	2	1	2	1	1
37*	24	1	1	1	2	2	1	1	1	88	55	2	1	1	2	1	2	2	2
38	63	1	1	1	2	2	1	2	2	72	57	2	1	1	2	1	1	2	1
39	10	1	1	1	2	2	2	2	1	73	38	2	1	1	2	1	1	1	2
40	11	1	1	1	2	2	2	1	2	74	44	1	2	1	2	1	2	1	1
41	65	2	2	1	2	2	1	1	1	75*	27	1	2	1	2	1	2	2	2
42	36	2	2	1	2	2	1	2	2	76*	61	1	2	1	2	1	2	2	2
43	47	2	2	1	2	2	2	2	1	77	60	1	2	1	2	1	1	2	1
44	25	2	2	1	2	2	2	1	2	78	28	1	2	1	2	1	1	1	2
45	43	2	1	2	2	2	1	1	1	79	58	1	1	2	2	1	2	1	1
46	29	2	1	2	2	2	1	2	2	80	17	1	1	2	2	1	2	2	2
47	46	2	1	2	2	2	2	2	1	81	56	1	1	2	2	1	1	2	1
48	1	2	1	2	2	2	2	1	2	82	34	1	1	2	2	1	1	1	2
49	32	1	2	2	2	2	1	1	1	83	18	2	2	2	2	1	2	1	1
50	14	1	2	2	2	2	1	2	2	84*	26	2	2	2	2	1	2	2	2
51	51	1	2	2	2	2	2	2	1	85*	23	2	2	2	2	1	2	2	2
52	15	1	2	2	2	2	2	1	2	86	41	2	2	2	2	1	1	2	1
53*	2	2	1	1	1	2	2	1	1	87	37	2	2	2	2	1	1	1	2

*Duplicate specimens

Factor Legend

A - Aggregate type
B - Aggregate gradation
C - Asphalt viscosity (specifications)
D - Asphalt content
E - Compaction type
F - Mixing temperature
G - Compaction temperature
H - Curing temperature

order of compaction used was the same as that used for mixing. This mixing-compaction sequence was required to maintain as closely as possible a constant curing time interval between the two processes. The order of testing was also randomized and can be seen in Table 7.

The total preparation and testing procedure is actually divided into the three distinct phases of (1) mixing, (2) compaction, and (3) curing. In the mixing phase five of the total number of factors are introduced into the experimental process. The error mean squares introduced during the mixing phase are then related to these five factors. Two more factors are added in the compaction phase, possibly adding errors associated with these new factors as well as the interactions of the added factors with the five factors included in the mixing phase. The final factor is then introduced in the curing phase and the errors in the experimental data at the completion of the overall process are then related to all eight factors. The relative magnitude of the errors introduced at the different phases governs the type of analysis required to evaluate the experimental data. If the errors associated with the individual phases remain relatively constant, the data can be analyzed as a completely randomized experiment. On the other hand if the error mean squares change from phase to phase, some type of split-plot analysis is required to evaluate the experimental data.

An example of the type of analysis required for the evaluation of such phasing is presented in Appendix 2. The technique used for the analysis assumes that the effects of certain three-way interactions are negligible and the sum of squares associated with them is attributable only to error. The assumption that these three-way interactions are zero is considered to be invalid by the authors since there are other three-way interactions which have highly significant effects upon the parameters evaluated in this study. Based upon this premise and preliminary checks, the assumption is made herein that the error mean squares remained constant throughout the three phases, allowing the completely randomized analysis to be valid.

Four pairs of duplicate specimens were utilized in the experimental designs and are indicated in Table 7. These duplicates were used to estimate the true error associated with the experimental preparation and testing of the specimens. This true estimate was subsequently used to calculate the F level of the fifty-four main effects and the two and three-way interactions of the eight factors under consideration.

The experiment was designed to allow all two-way interactions to be estimated, that is, no smaller than a three-way interaction confounded with them. There were also 18 three-way interactions which had only four-way or higher interactions confounded with them. The major effects and interactions analyzed in this experiment are outlined in Table 8.

TABLE 8. FACTORS AND INTERACTIONS ANALYZED IN EXPERIMENT

Two-Way Interactions		Three-Way Interactions	
A×B	C×E	A×D×F	B×E×F
A×C	C×F	A×D×G	B×E×G
A×D	C×G	A×D×H	B×E×H
A×E	C×H	A×E×F	C×D×F
A×F	D×E	A×E×G	C×D×G
A×G	D×F	A×E×H	C×D×H
A×H	D×G	B×D×F	C×E×F
B×C	D×H	B×D×G	C×E×G
B×D	E×F	B×D×H	C×E×H
B×E	E×G		
B×F	E×H		
B×G	F×G		
B×H	F×H		
C×D	G×H		

Main Factors

A - Aggregate type
B - Aggregate gradation
C - Asphalt viscosity (specification)
D - Asphalt content
E - Compaction type
F - Mixing temperature
G - Compaction temperature
H - Curing temperature

The statistical analysis included an analysis of variance of each of the dependent variables previously discussed. The analysis of variance provided the method of determining the significance and order of significance of fifty-four main effects and interactions included in this study. A regression analysis was completed on those factors and interactions determined to be significant at a level of 0.05 or greater by the analysis of variance. The output was predictive equations which can be used to estimate the tensile strength and horizontal failure deformation value for any combination of the original eight factors.

CHAPTER 5. EXPERIMENTAL RESULTS

The tensile properties of an asphalt-treated material can be characterized by its tensile strength and tensile strain. The test parameters analyzed in this study were selected to approximate these tensile properties as closely as possible.

The indirect tensile strength values obtained in this experiment are based upon a simple equation which assumes that there is no effect due to Poisson's ratio on the stabilized material. This assumption may be questionable since a multiaxial state of stress actually exists in the specimen. At the present time, however, there is no development available for this test from which Poisson's ratio can be obtained without the extensive use of strain gages. The cost and difficulty involved in attaching strain gages to asphalt-treated mixtures made this approach undesirable; therefore, the effect of Poisson's ratio was neglected.

The horizontal failure deformation value analyzed in this experiment is the summation of strains produced along the horizontal axis by both the radial and tangential stresses (see Fig 5) and is some measure of the tensile strain at failure of the specimen. It must be noted, however, that the Poisson's ratio of the material must also be known before the actual strain due to the tensile stress at the center can be calculated.

The values of the two test parameters analyzed in this study are presented in Table 9. An analysis of variance was completed on these two dependent variables. The main effects, two-way interactions, and three-way interactions were then ordered, beginning with the most significant factor or interaction and listing each of the remaining in descending order of significance. In each analysis the lowest numbered specimen of the duplicates was used in the analysis of variance, that is, 36, 53, 75, and 84. The true error mean squares estimate was then obtained using the data from the duplicate specimen and was used to calculate the significance level of each of the main effects and interactions.

TABLE 9. EXPERIMENTAL RESULTS

Specimen Number	Indirect Tensile Strength	Horizontal Failure Deformation	Specimen Number	Indirect Tensile Strength	Horizontal Failure Deformation
20	74.5	.0100	55	23.9	.0076
21	108.4	.0064	56	13.5	.0154
22	105.1	.0056	57	30.9	.0098
23	65.5	.0074	58	41.5	.0102
24	60.7	.0098	59	82.8	.0064
25	62.9	.0136	60	65.6	.0078
26	80.5	.0072	61	54.6	.0136
27	35.9	.0080	62	50.8	.0078
28	29.8	.0144	63	25.5	.0034
29	37.0	.0052	64	83.0	.0062
30	15.6	.0073	90	30.2	.0070
31	14.3	.0088	66	29.2	.0068
32	95.7	.0112	67	53.5	.0102
89	111.3	.0164	68	43.6	.0064
34	148.5	.0040	69	43.2	.0064
35	82.9	.0068	70	120.4	.0068
36*	91.5	.0184	88	191.3	.0056
37*	82.3	.0218	72	169.4	.0068
38	127.8	.0176	73	117.1	.0126
39	156.3	.0114	74	82.8	.0190
40	133.5	.0178	75*	124.2	.0132
41	69.4	.0210	76*	126.7	.0140
42	137.7	.0093	77	73.4	.0262
43	134.3	.0106	78	55.5	.0228
44	120.1	.0077	79	149.0	.0176
45	131.2	.0122	80	231.3	.0090
46	185.0	.0051	81	179.8	.0170
47	158.7	.0044	82	78.8	.0282
48	166.5	.0068	83	129.2	.0128
49	85.8	.0254	84*	195.1	.0052
50	122.5	.0190	85*	204.9	.0065
51	148.1	.0124	86	116.7	.0180
52	125.0	.0180	87	107.2	.0072
53*	7.6	.0090			
54*	12.8	.0092			

*Duplicate specimens.

During the experiment it was necessary to replace specimen numbers 33, 65, and 71 with replacement specimen numbers 89, 90, and 88, respectively. During the testing phase the load-deformation data for specimens 33 and 65 did not plot properly on the x-y plotter and required substitute specimens. During the preparation phase an incorrect amount of asphalt content was added to the aggregate mix for specimen No. 71; therefore, a replacement specimen with the correct asphalt content was required. A detailed explanation of the experimental procedure from preparation to testing is presented in Appendix 3.

CHAPTER 6. DISCUSSION OF RESULTS

The principal objective of this study was to evaluate the factors affecting the tensile properties of asphalt-treated materials. In an attempt to make observed differences as large as possible, two relatively extreme levels of the factors were utilized in making the evaluation. The experiment was then designed to evaluate the effects of the main factors and some of their interactions. It was not intended that this study provide a detailed investigation of the mechanisms which produce the effects.

Those factors or interactions found to be significant at alpha levels of 0.01 and 0.05 for each dependent variable are presented in Tables 10 and 11; all other factors and interactions were considered not to be significant.

The relationships of the significant main factors and their interactions with the dependent variables of tensile strength and horizontal failure deformation are presented in Figs 12 through 35 and Figs 36 through 43, respectively. The data points presented in these figures are the average values of the dependent variables for all specimens containing a given level or combination of levels of the main factor or interaction. For instance, each plotted point for a main factor is the mean value obtained from the 32 specimens which included that particular level of the factor. There are four possible combinations of factor levels for a two-way interaction; therefore, each value plotted is the mean for the data from sixteen different specimens. A three-way interaction has eight possible combinations of factor levels; therefore, each of the points portrayed for the interaction is the mean value of the results of eight specimens.

STATISTICAL INFERENCES

Since two and three-way interactions were found to be significant, the results have meaning for the investigator as shown in the corresponding figures. In other words, the variation of tensile strength values other than that due to main effects observed among the various combinations of the factors

TABLE 10. ANALYSIS OF VARIANCE FOR TENSILE STRENGTH

Source of Variation	Degrees of Freedom	Mean Squares	F Value	Significance Level, %
D	1	90420.0	3380.0	1
G	1	15661.2	585.5	1
B×D	1	13716.4	512.8	1
A×D	1	13427.6	502.0	1
D×F	1	5590.0	209.0	1
E	1	3538.8	132.3	1
F	1	2945.6	110.1	1
C×D	1	2835.8	106.0	1
D×E	1	2786.5	104.2	1
C	1	2332.2	87.2	1
D×G	1	2189.8	81.9	1
A	1	2030.6	75.9	1
B×C	1	1649.4	61.7	1
D×H	1	1446.5	54.1	1
E×G	1	1401.4	52.4	1
A×H	1	1245.6	46.6	1
B×D×G	1	829.7	31.0	1
A×C	1	785.3	29.4	1
B×E×G	1	708.6	26.5	1
A×E	1	676.9	25.3	1
C×E×F	1	644.9	24.1	1
A×B	1	618.2	23.1	1
A×F	1	592.7	22.2	1
A×G	1	570.2	21.3	1
B	1	520.0	19.4	5
B×F	1	512.8	19.2	5
E×H	1	429.8	16.1	5
E×F	1	355.1	13.3	5
H	1	303.0	11.3	5
A×D×G	1	226.0	8.5	5
C×D×H	1	217.6	8.1	5
G×H	1	206.9	7.7	5
Within treatments treated alike	4	26.8	---	-

Legend of Factors

A - Aggregate type
B - Aggregate gradation
C - Asphalt viscosity
D - Asphalt content
E - Compaction type
F - Mixing temperature
G - Compaction temperature
H - Curing temperature

TABLE 11. ANALYSIS OF VARIANCE FOR HORIZONTAL FAILURE DEFORMATION

Source of Variation	Degrees of Freedom	Mean Squares ($\times 10^{-4}$)	F Value	Significance Level, %
D	1	4.463	256.3	1
A×D	1	3.832	220.1	1
F	1	2.706	155.4	1
A	1	2.449	140.7	1
B×D×H	1	1.243	71.4	1
G	1	1.035	59.5	1
B	1	.640	36.8	1
B×E×G	1	.533	30.6	1
D×G	1	.325	18.7	5
D×F	1	.322	18.5	5
C×E	1	.305	17.5	5
A×H	1	.284	16.3	5
B×G	1	.248	14.2	5
H	1	.214	12.3	5
A×G	1	.214	12.3	5
B×D	1	.214	12.3	5
A×F	1	.189	10.9	5
D×H	1	.166	9.7	5
E×F	1	.154	8.9	5
C	1	.123	7.0	5
Within treatments treated alike	4	.017	---	-

Legend of Factors

A - Aggregate type
B - Aggregate Gradation
C - Asphalt viscosity
D - Asphalt content
E - Compaction type
F - Mixing temperature
G - Compaction temperature
H - Curing temperature

was not random but was due to some relationship among the factors. If the investigator desires to infer to a specific combination of factors, it is not adequate to include only the main effects of the factors. He must also consider the interaction among the factors to predict the results with precision. This is an important point and means that small isolated experiments can give misleading results if several important factors are held constant, because the results can not accurately be inferred to other levels of that variable and important interactions can be missed. For example, suppose Newton in his work had performed an experiment to investigate the relationship of force, mass, and acceleration. In selecting a single mass for his experiment he would have obtained the equation $F = C_1 a$. In an experiment with a single level of acceleration he would have obtained $F = C_2 m$. However, in a complete experiment he obtained $F = ma$ which contains "all interaction" and does not contain a term for mass or acceleration alone. The relationship is "all interaction" because the rate of change of force with respect to mass is dependent upon the level of magnitude of acceleration and vice versa. In general, the significant three-way interactions should be explained in order for two-way interactions to have much practical meaning and in turn the two-way interactions should be discussed before the main effects for best understanding.

INDIRECT TENSILE STRENGTH

Table 10 shows that the number of factors and interactions affecting the tensile strength of asphalt-treated materials was quite large, with 32 out of a possible 54 combinations being significant at levels of 5 percent or greater. However, not all of these effects had practical significance. In other words the effect, although measurable, was not large and probably would make no effective difference in the application of the results. Since the results of the experiment were to be used by engineers, the statistical explanation needed to be in terms of engineering application. In order for the engineer to fully comprehend their practical significance, only those main effects and interactions significant at the 1 percent probability level were considered to have practical meaning. In the following sections, the highly significant three-way interactions, two-way interactions, and main effects are discussed in order.

An example of the importance of interactions can be found by reviewing the highly significant effects portrayed in Table 10. The two main effects which were not considered of practical significance ($\alpha = 0.01$) were gradation (Factor B) and curing temperature (Factor H). If only main effects were considered in this experiment, the two effects would probably not be included in future studies. From the study of interactions, however, it was found that there were two three-way interactions and two two-way interactions involving gradation (Factor B) and two two-way interactions involving curing temperature (Factor H) which were of practical significance. The two main effects then must be considered in future experiments because of their important interaction effects.

Three-Way Interactions

Five three-way interactions were found to significantly affect the tensile strength of asphalt-treated materials at a probability level of 0.05. The three multiple interactions which were significant at the 0.01 level are presented in Figs 12 through 14 and are discussed in the following paragraphs. The other two interactions were considered to be of no practical significance to engineers.

Aggregate Gradation × Asphalt Content × Compaction Temperature (Interaction B×D×G). The three-way interaction, shown in Fig 12, between gradation, asphalt content, and compaction temperature had a significant effect on tensile strength. The lowest strength value was obtained for a fine graded material with 3.5 percent asphalt which was compacted at a temperature of 200° F. The greatest tensile strength for these data occurred for a fine graded material at an asphalt content of 7.0 percent compacted at 300° F. The slopes and orientation of surfaces formed by connecting the tops of the bars for the two gradation types indicated the effects of total interaction on the tensile strength of a treated mixture. The surface generally sloped upward from the point of low asphalt and low compaction temperature to the point of highest asphalt content and high compaction temperature. At the higher asphalt contents the specimens composed of fine aggregate gradations exhibited higher tensile strengths while at low asphalt contents, mixtures containing coarse aggregate gradation exhibited greater strength. From this three-dimensional plot, it is possible to see the effect of any combination of these three factors upon the tensile strength.

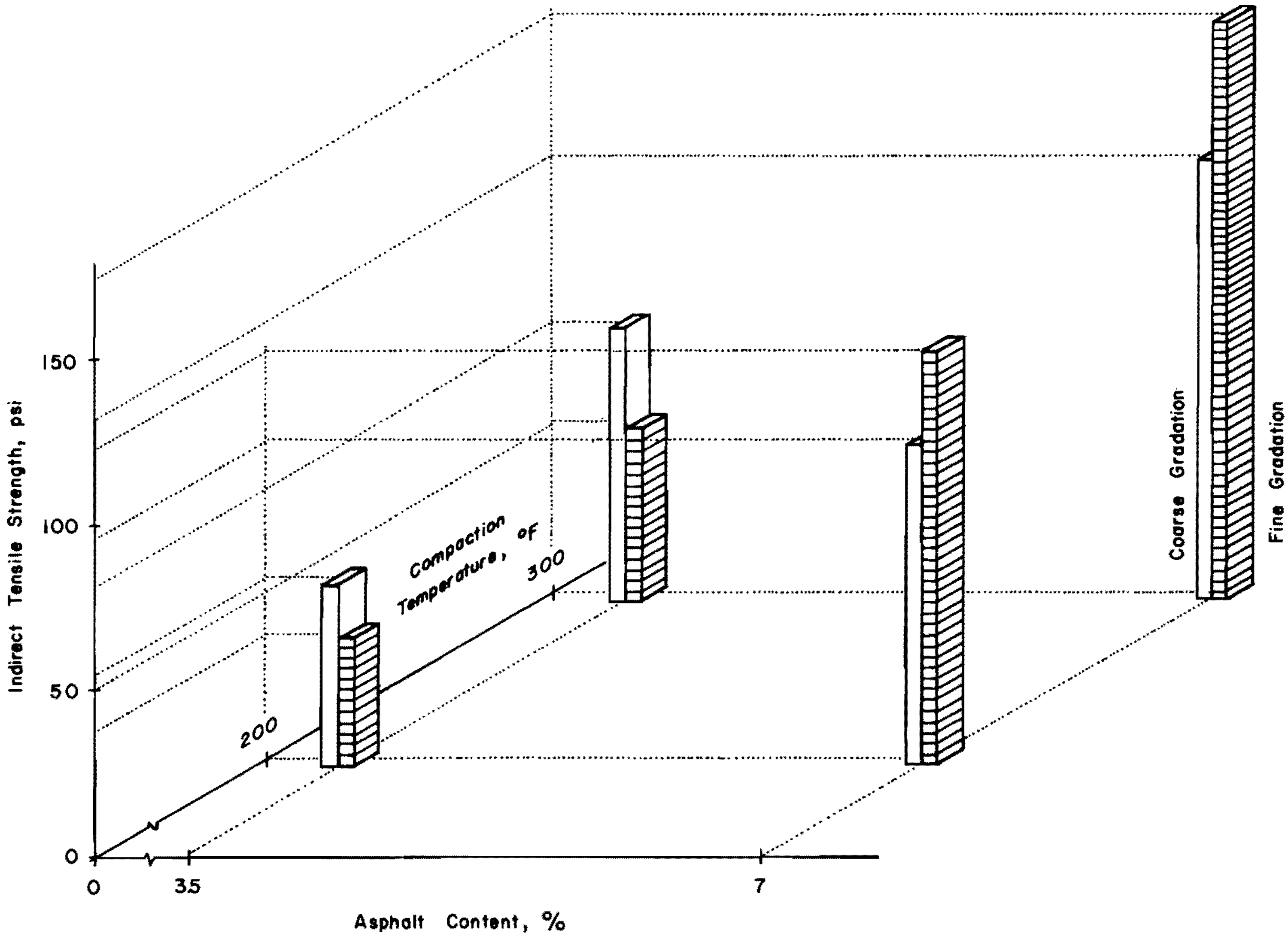

Fig 12. Effect of interaction between aggregate gradation, asphalt content, and compaction temperature on tensile strength (Interaction B×D×G).

Gradation Type × Compaction Type × Compaction Temperature (Interaction B×E×G). The next three-way interaction which affected the tensile strengths involved gradation type, compaction type, and compaction temperature. A three-dimensional plot showing the relationship is presented in Fig 13. The combination of 300° F compaction temperature, impact compaction, and fine gradation produced the greatest tensile strengths while the lowest strength values were obtained for the combination of 200° F compaction temperature, impact compaction, and coarse gradation.

Asphalt Viscosity × Compaction Type × Mixing Temperature (Interaction C×E×F). The results of the analysis of variance also indicated that the three-way interaction between compaction type, mixing temperature, and asphalt viscosity was important in determining the tensile strength of asphalt-treated materials. The three-dimensional plot presented in Fig 14 shows the effects of this interaction. From the figure it can be seen that the combination of the three-way interaction involving the more viscous asphalt cement (AC-20) produced stronger specimens.

Two-Way Interactions

In this study nineteen two-way interactions were shown to be significant at a level of 0.05 or greater. Fifteen of these were also found to be highly significant at a level of 0.01 with 12 having practical significance for the engineer.

The strength effects produced by these 15 interactions are summarized in Figs 15 through 29 and a brief description of the 12 effects having practical significance is included below.

Aggregate Gradation × Asphalt Content (Interaction B×D). Asphalt content and aggregate gradation interacted significantly as shown in Fig 15. A much greater increase in tensile strength occurred with an increase in asphalt content for specimens composed of finely graded aggregates than for specimens composed of coarse graded aggregates. At the low asphalt content, maximum strength occurred with the coarse gradation, while at the high asphalt content, maximum strength occurred with the fine gradation.

Aggregate Type × Asphalt Content (Interaction A×D - Fig 16). It was found that asphalt content had a greater effect on the strength of specimens containing rounded Seguin gravel than those composed of limestone. At an

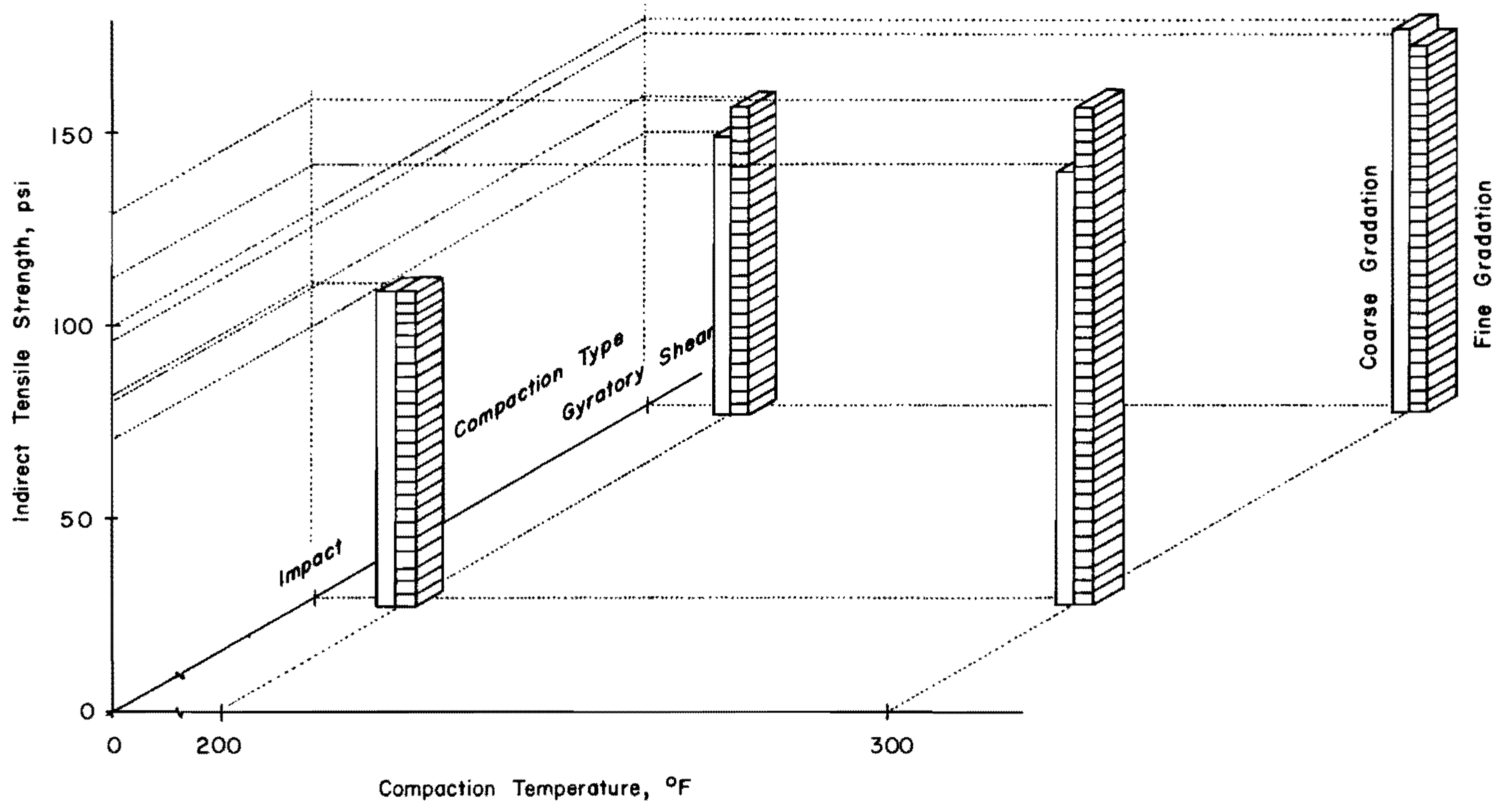

Fig 13. Effect of interaction between aggregate gradation, compaction type, and compaction temperature on tensile strength (Interaction B×E×G).

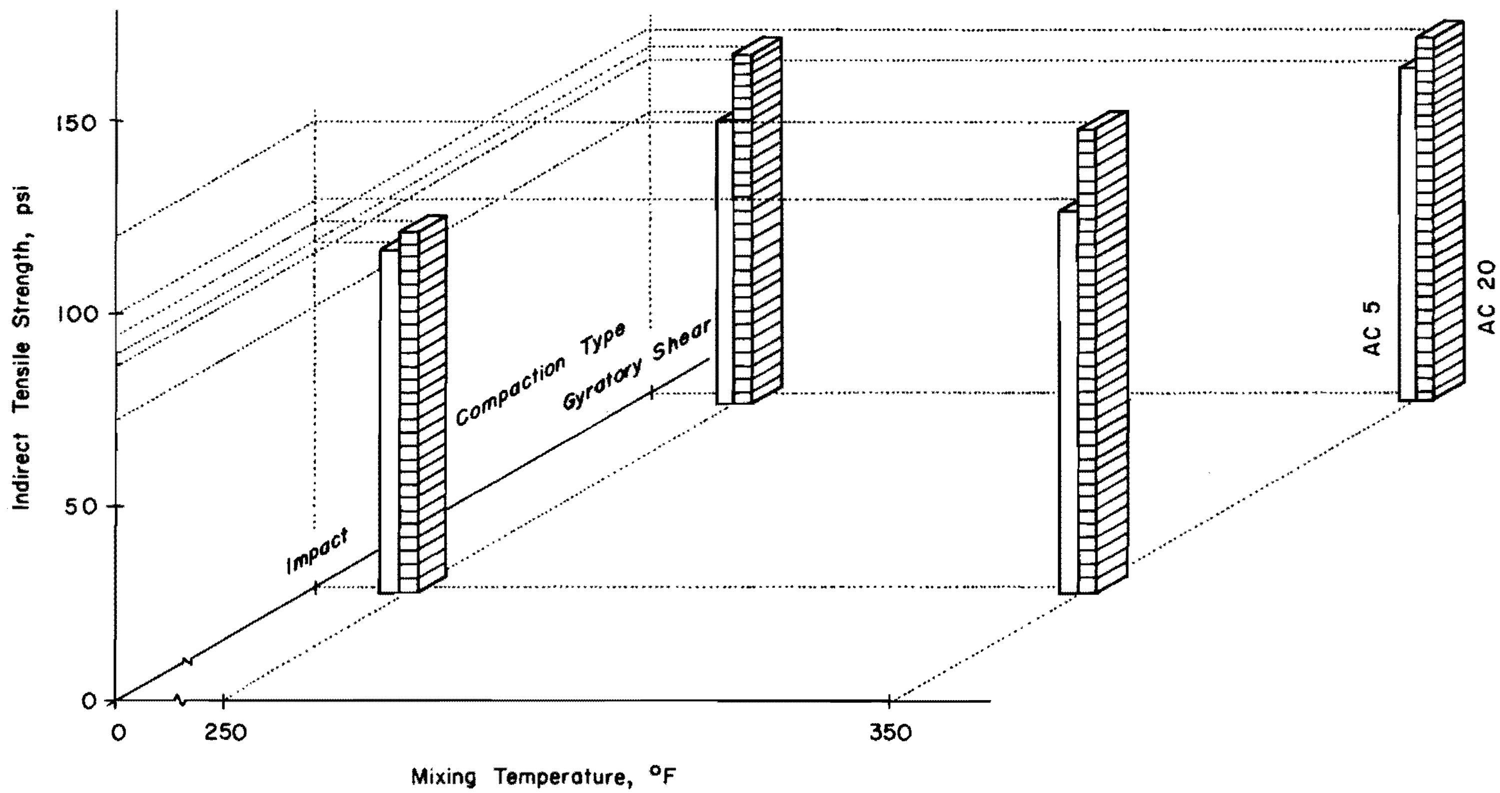

Fig 14. Effect of interaction between asphalt viscosity, compaction type, and mixing temperature on tensile strength (Interaction C×E×F).

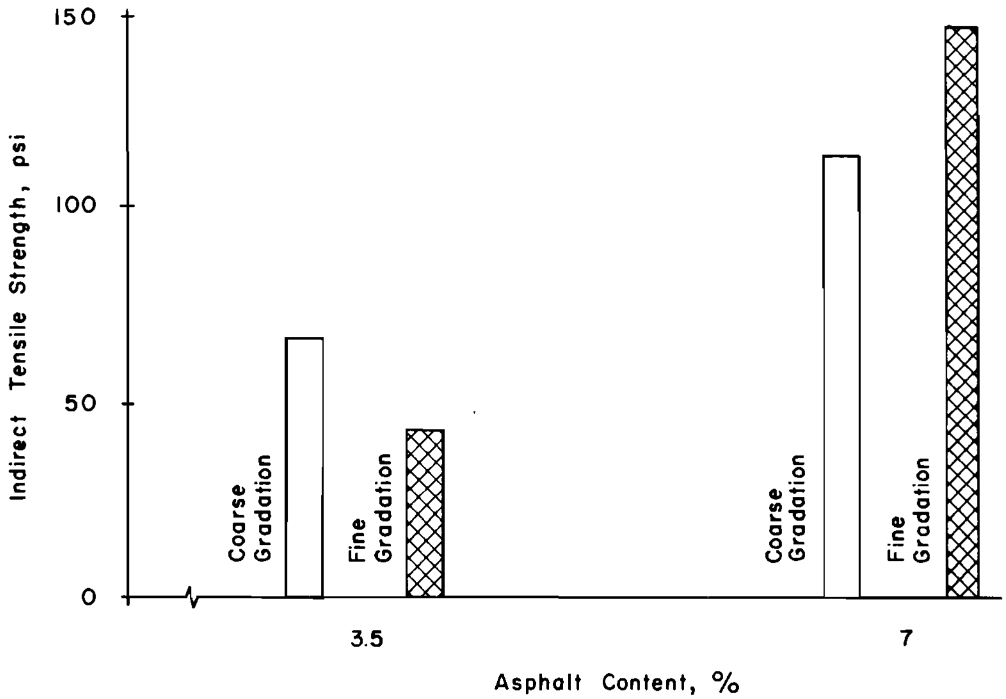

Fig 15. Effect of interaction between aggregate gradation and asphalt content on tensile strength (Interaction B×D).

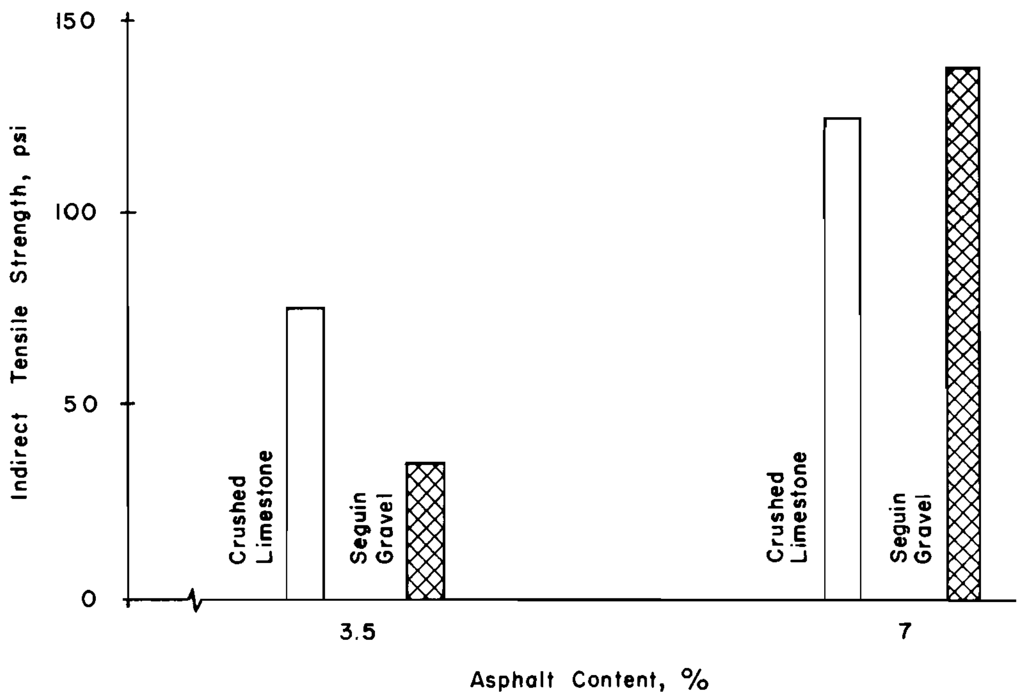

Fig 16. Effect of interaction between aggregate type and asphalt content on tensile strength (Interaction A×D).

asphalt content of 3.5 percent the limestone specimens exhibited greater tensile strength while at 7.0 percent the Seguin gravel specimens were stronger.

Asphalt Content × Mixing Temperature (Interaction D×F). It can be seen in Fig 17 that at the low asphalt content there was very little difference in the strength values for the two mixing temperatures; however, at an asphalt content of 7.0 percent the average strength for specimens mixed at 350° F was much greater than the average strength of specimens mixed at 250°F.

Asphalt Viscosity × Asphalt Content (Interaction C×D). Figure 18 shows the two-way interaction of asphalt viscosity with asphalt content for average tensile strength. The strength at the low asphalt content was essentially the same for both asphalt viscosities while at the high asphalt content the strength of specimens composed of the more viscous asphalt (AC-20) was greater than for the less viscous asphalt (AC-5).

Asphalt Content × Compaction Type (Interaction D×E - Fig 19). At a low asphalt content the type of compaction appeared to be important, with the impact compaction method producing specimens with higher tensile strengths. At the higher asphalt content, however, the average strengths were in close agreement.

Asphalt Content × Compaction Temperature (Interaction D×G - Fig 20). Compaction temperature and asphalt content interacted to produce a significant effect on strength. It can be seen that the increase in strength due to increased compaction temperature was greater at the higher asphalt content.

Aggregate Gradation × Asphalt Viscosity (Interaction B×C - Fig 21). For the finely graded aggregate, a change in asphalt viscosity produced no change in strength; however, for the coarse graded aggregate an increase in viscosity did result in a slight increase in strength.

Asphalt Content × Curing Temperature (Interaction D×H). As shown in Fig 22 the curing temperature of 110° F produced a greater increase in tensile strength over the range of asphalt contents than did the lower curing temperature of 40° F.

Compaction Type × Compaction Temperature (Interaction E×G). As shown in Fig 23 those specimens compacted by the impact method displayed strengths greater than those by the gyratory-shear method, while the effect of

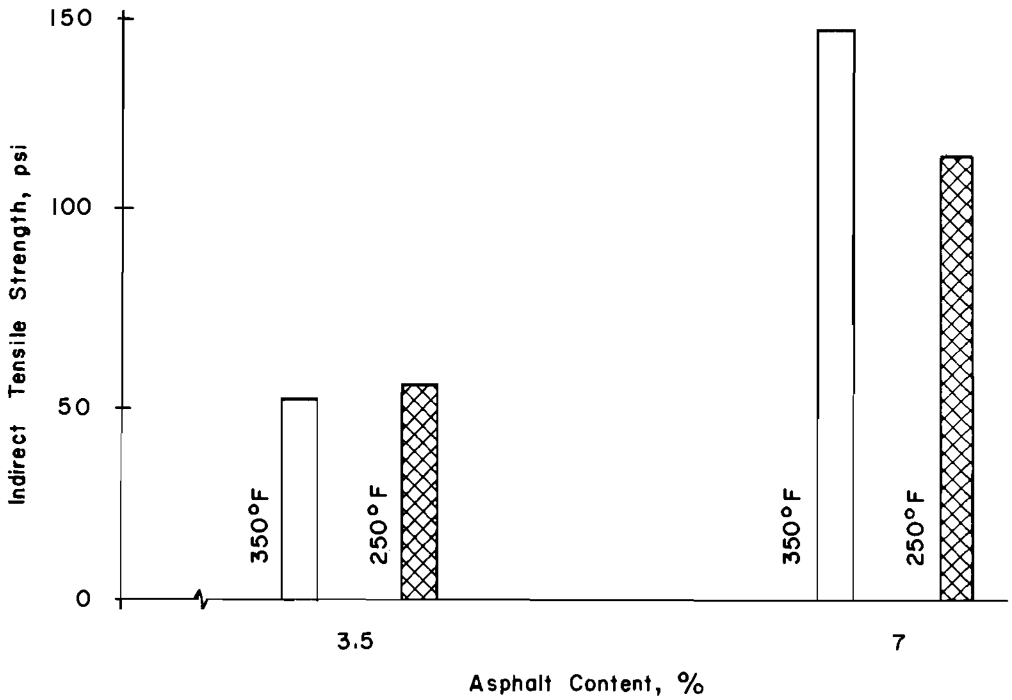

Fig 17. Effect of interaction between asphalt content and mixing temperature on tensile strength (Interaction D×F).

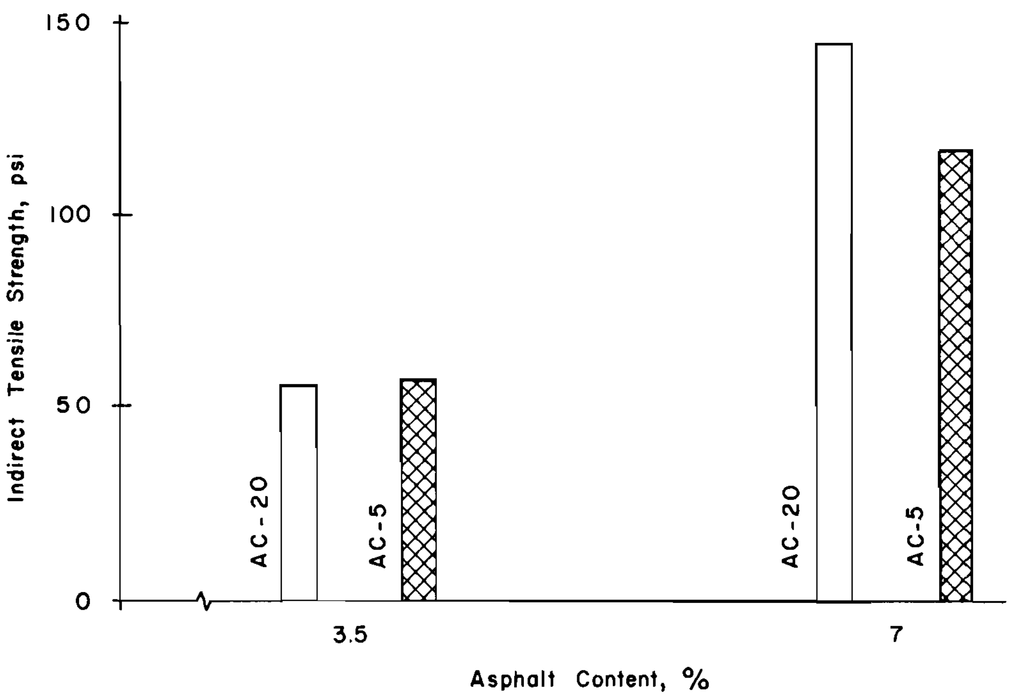

Fig 18. Effect of interaction between asphalt viscosity and asphalt content on tensile strength (Interaction C×D).

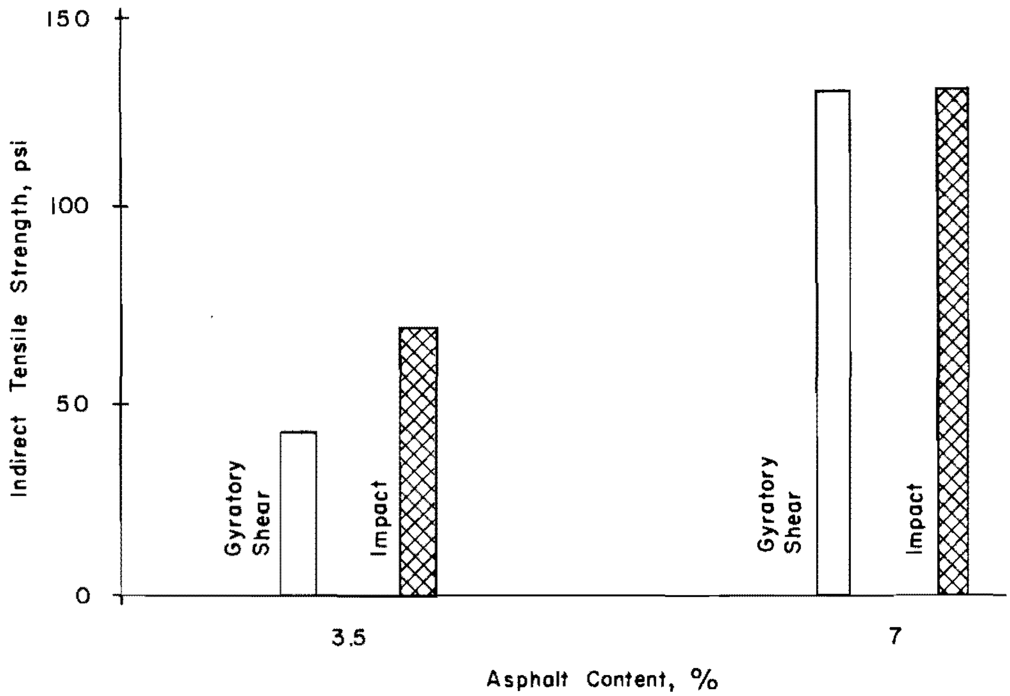

Fig 19. Effect of interaction between asphalt content and compaction type on tensile strength (Interaction D×E).

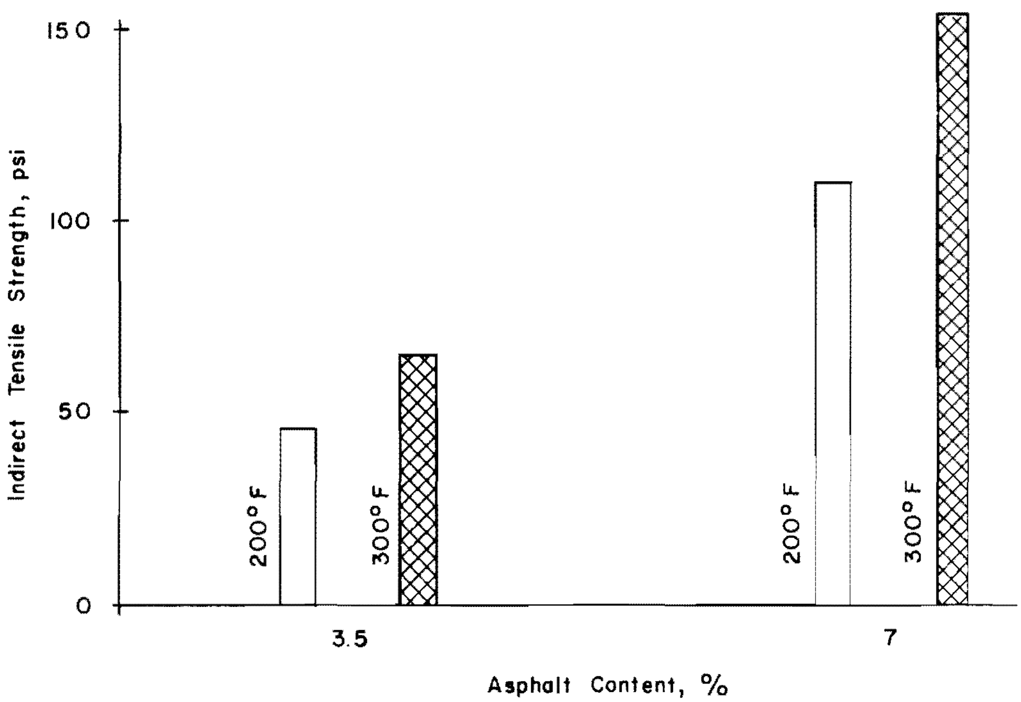

Fig 20. Effect of interaction between asphalt content and compaction temperature on tensile strength (Interaction D×G).

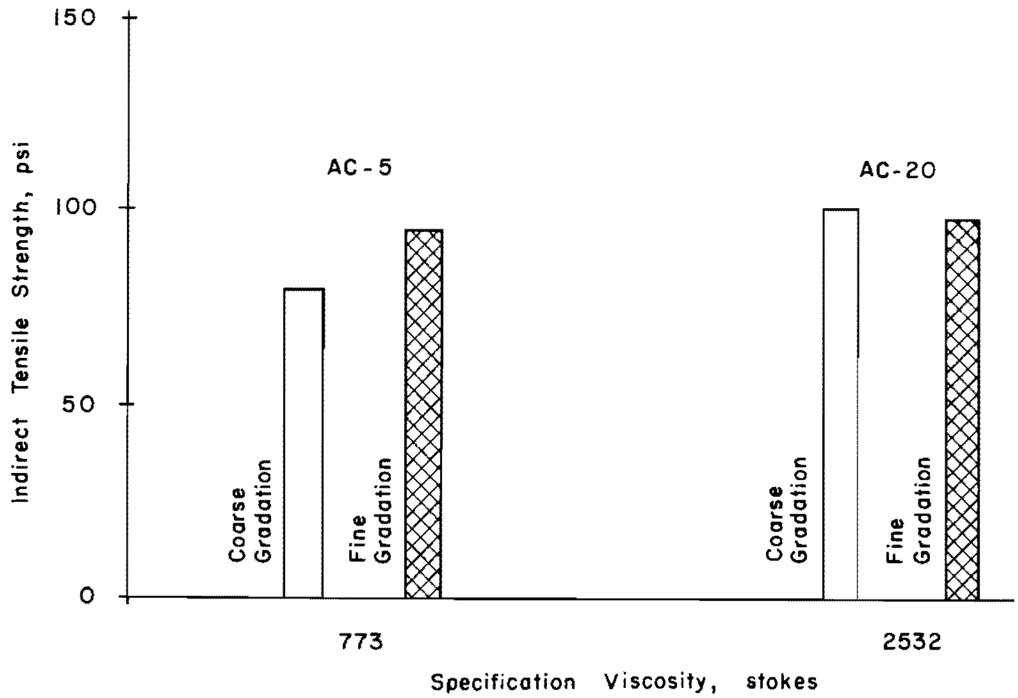

Fig 21. Effect of interaction between aggregate gradation and asphalt viscosity on tensile strength (Interaction B×C).

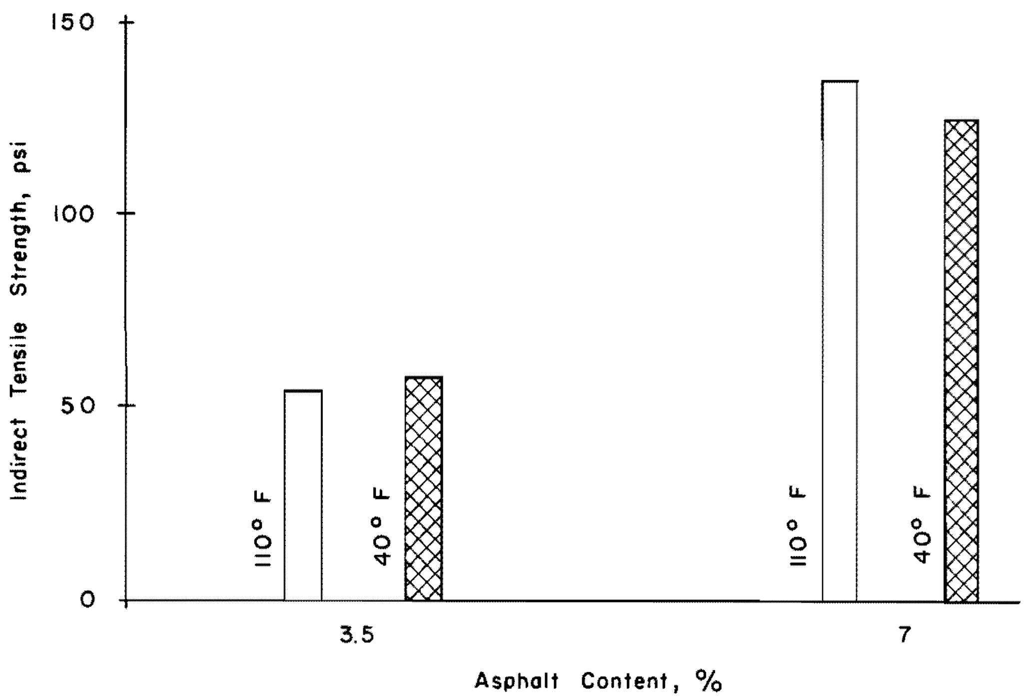

Fig 22. Effect of interaction between asphalt content and curing temperature on tensile strength (Interaction D×H).

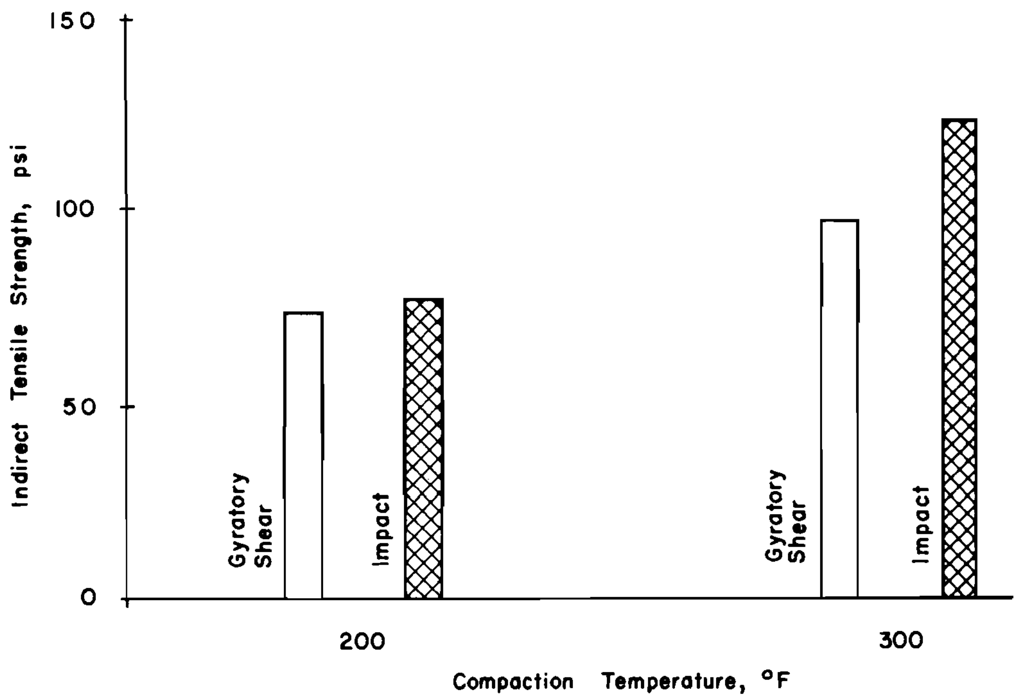

Fig 23. Effect of interaction between compaction type and compaction temperature on tensile strength (Interaction E×G).

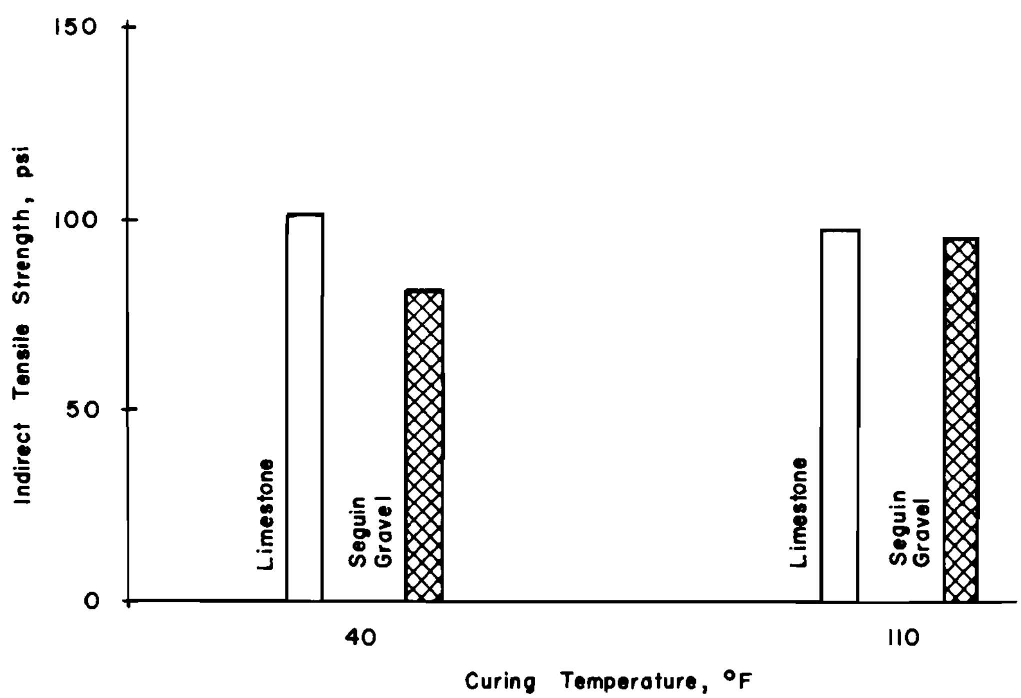

Fig 24. Effect of interaction between aggregate type and curing temperature on tensile strength (Interaction A×H).

increasing compaction temperature was greater upon those specimens compacted by the impact method.

Aggregate Type × Curing Temperature (Interaction A×G - Fig 24). Although the analysis of variance indicated that this interaction was significant at the one percent level, the difference in the strengths for asphalt-treated limestone materials at the two curing temperatures was small. The specimens composed of asphalt-treated Seguin gravel, however, did exhibit somewhat greater strengths at the higher curing temperature.

Aggregate Type × Asphalt Viscosity (Interaction A×C - Fig 25). The tensile strength was significantly affected by the interaction of aggregate type and asphalt viscosity. Regardless of the viscosity of the asphalt used, the specimens containing limestone aggregate were stronger than those with rounded gravel. For both aggregate types the increase in asphalt viscosity also increased the tensile strength of the asphalt-treated materials.

Aggregate Type × Compaction Type (Interaction A×E). As shown in Fig 26, the effect of changing compaction methods was greater for limestone specimens than for those made of rounded gravel.

Remaining Two-Way Interactions. The remaining two-way interactions are depicted in Figs 27, 28, and 29; they were judged to be of no practical significance and, therefore, will not be discussed.

Main Effects

All eight main factors were found by analysis of variance to be significant at a level of 0.05 or greater; in addition, six of these eight factors were found to be highly significant at a level of 0.01. These six factors also showed practical significance. Only aggregate gradation and curing temperature failed to produce highly significant effects on the indirect tensile strength.

Figures 30 through 35 graphically summarize the effects produced by the six highly significant factors. From these figures it can be seen that the average indirect tensile strength was increased significantly by

(1) increasing the asphalt content from 3.5 percent to 7.0 percent (Fig 30),

(2) increasing the compaction temperature from 200 to 300° F (Fig 31),

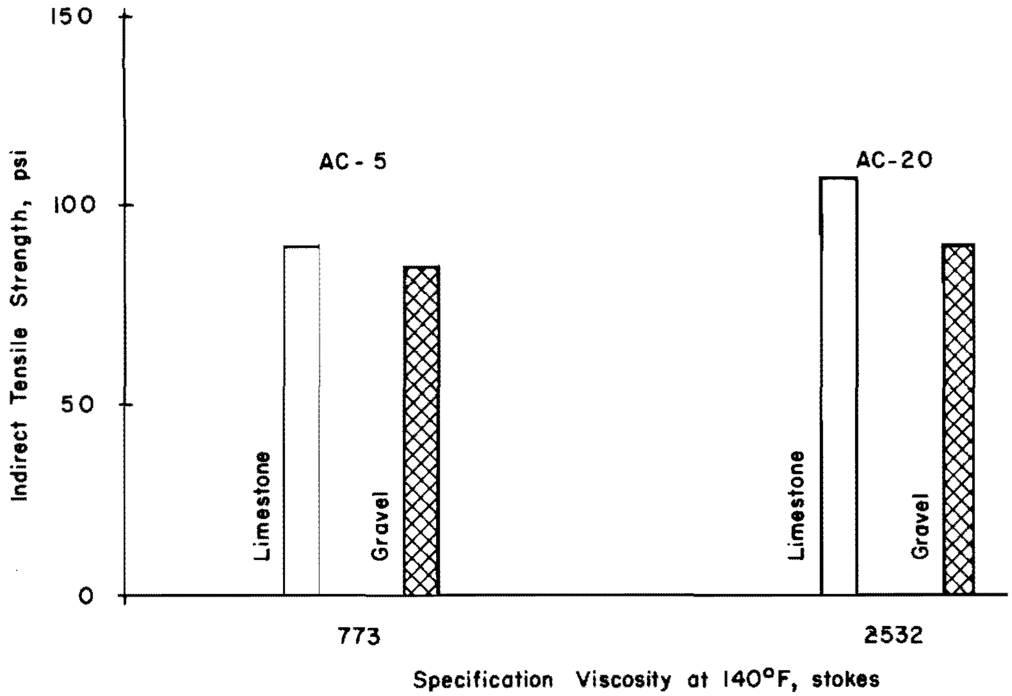

Fig 25. Effect of interaction between aggregate type and asphalt viscosity on tensile strength (Interaction A×C).

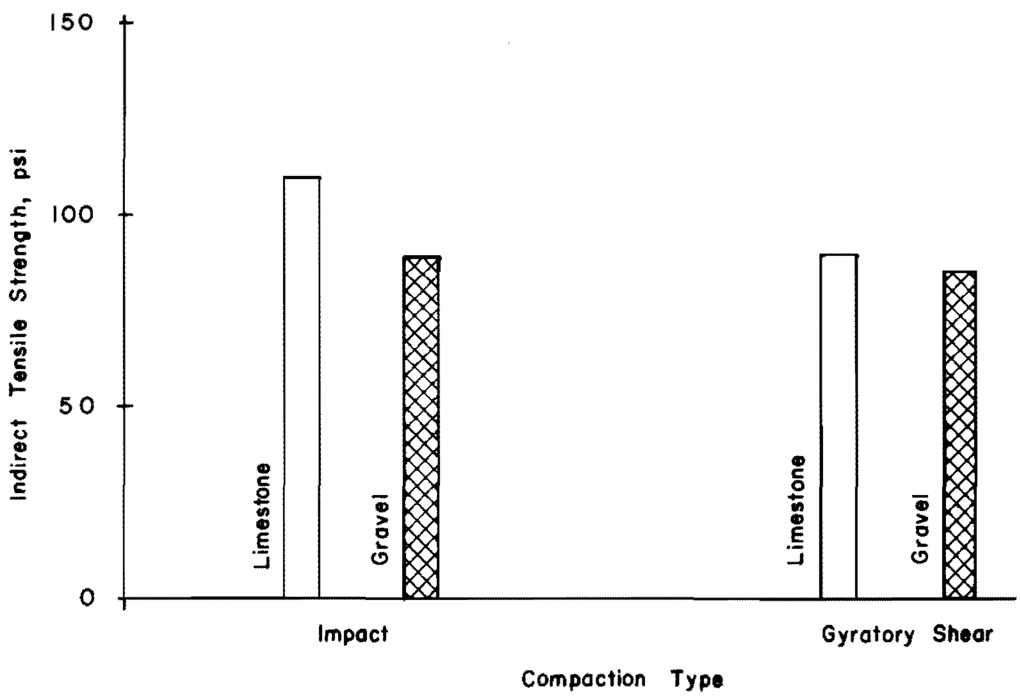

Fig 26. Effect of interaction between aggregate type and compaction type on tensile strength (Interaction A×E).

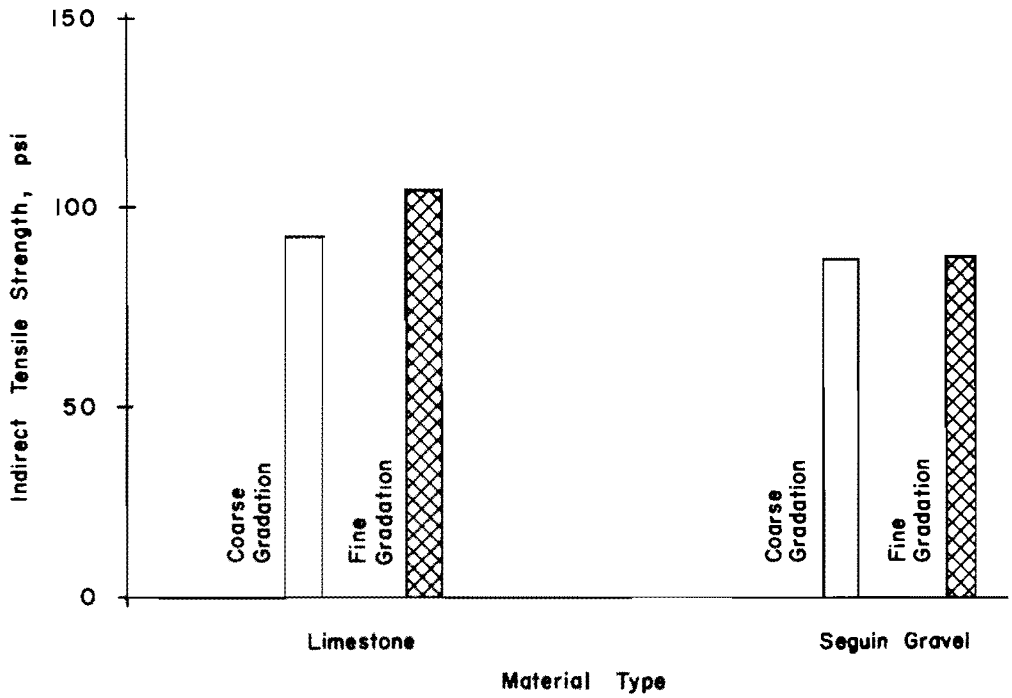

Fig 27. Effect of interaction between aggregate type and aggregate gradation on tensile strength (Interaction A×B).

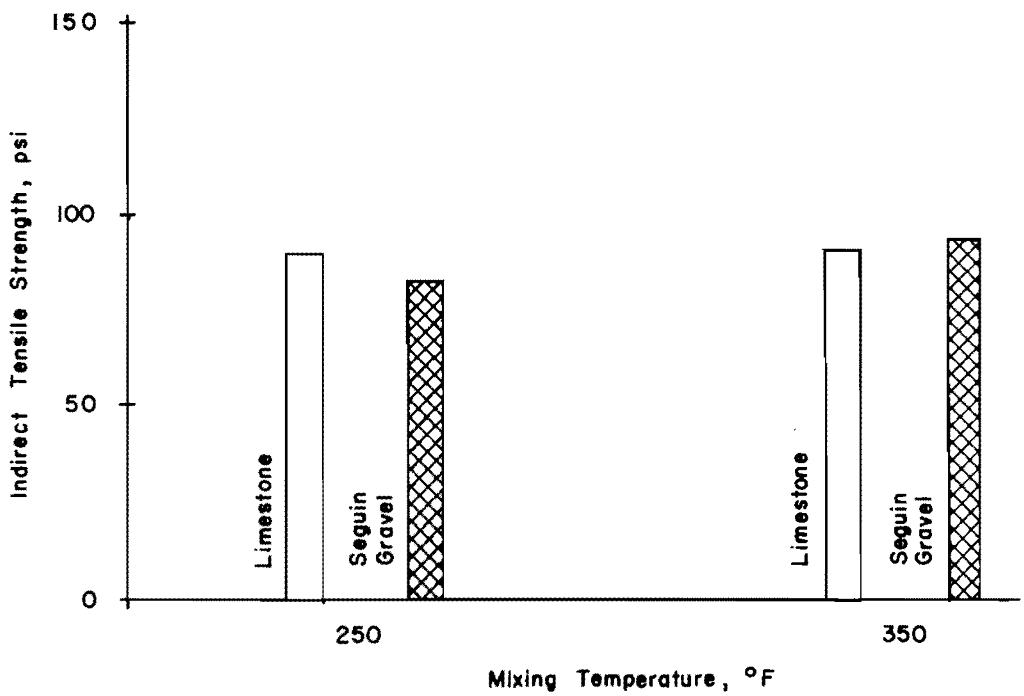

Fig 28. Effect of interaction between aggregate type and mixing temperature on tensile strength (Interaction A×F).

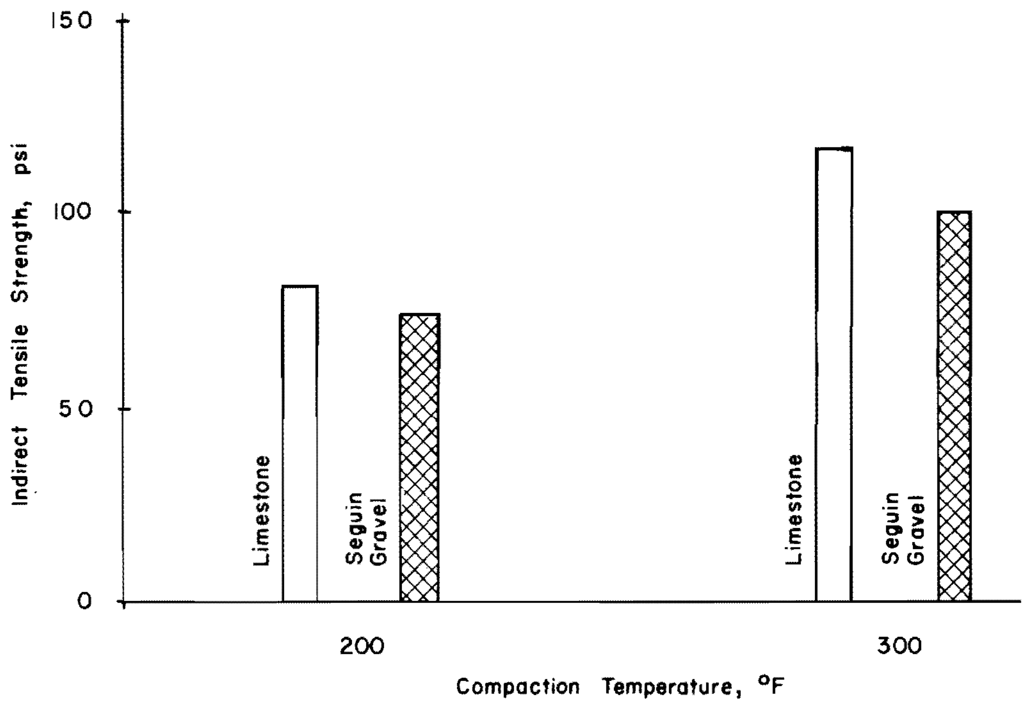

Fig 29. Effect of interaction between aggregate type and compaction temperature on tensile strength (Interaction A×G).

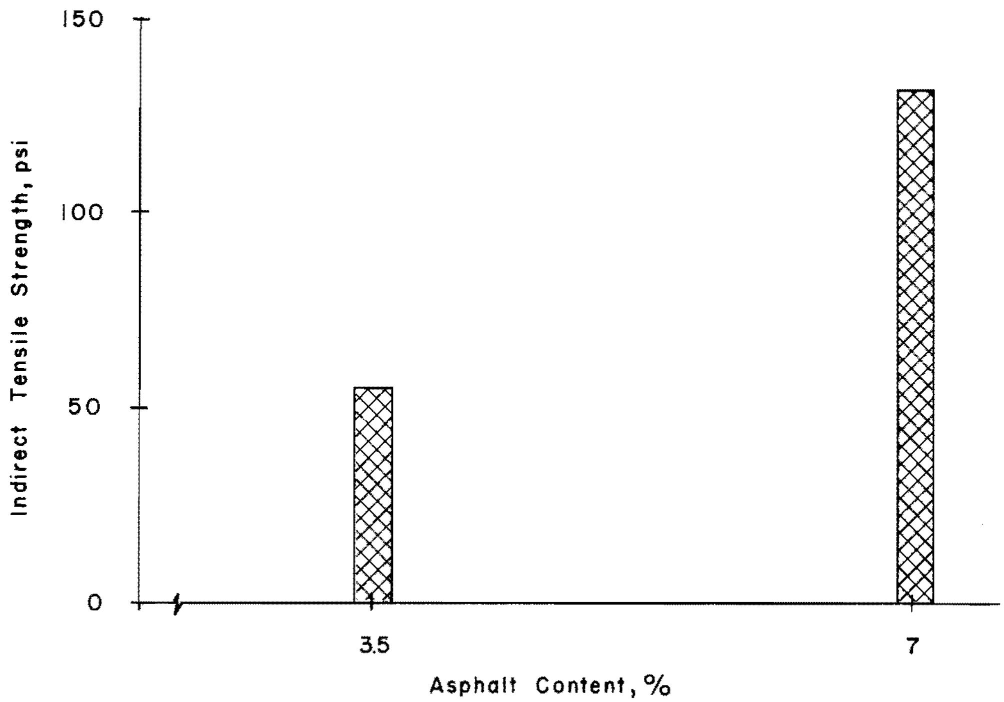

Fig 30. Effect of asphalt content on tensile strength (Factor D).

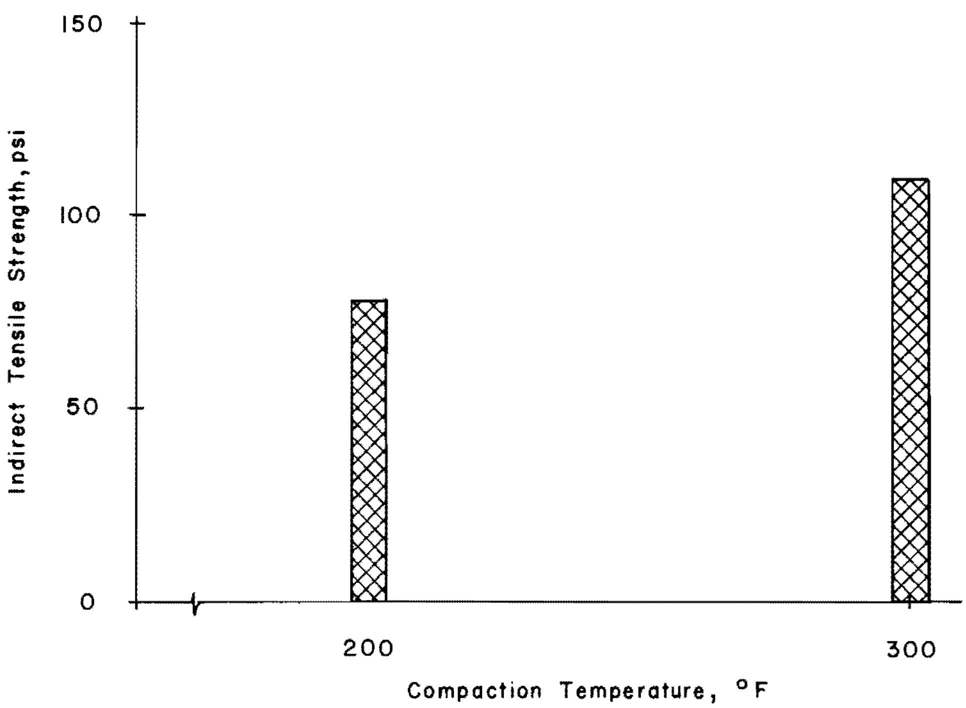

Fig 31. Effect of compaction temperature on tensile strength (Factor G).

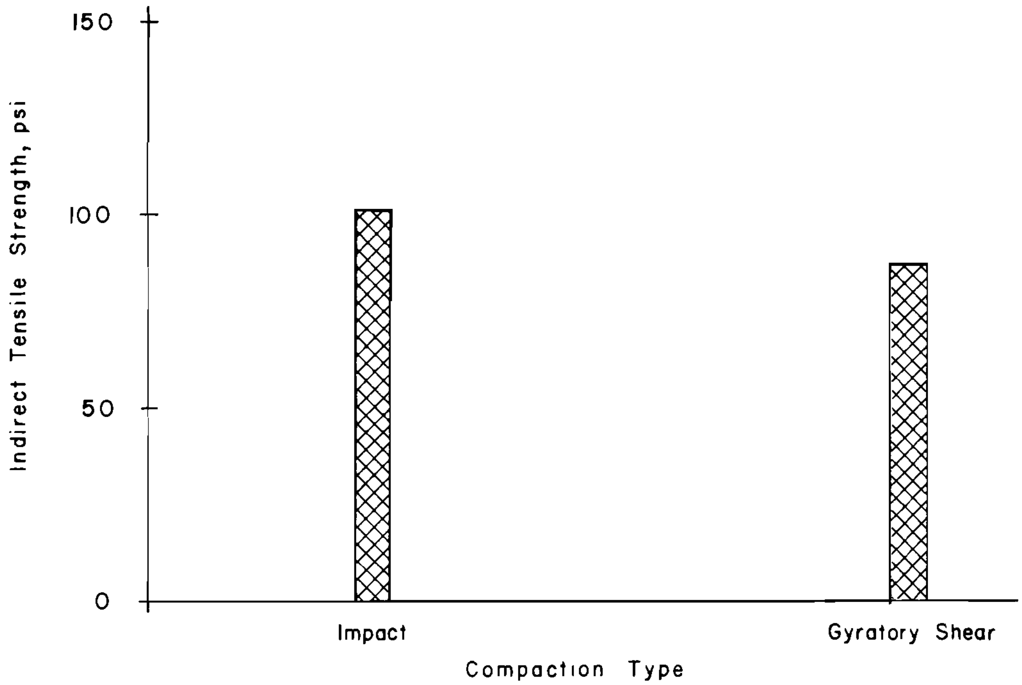

Fig 32. Effect of compaction type on tensile strength (Factor E).

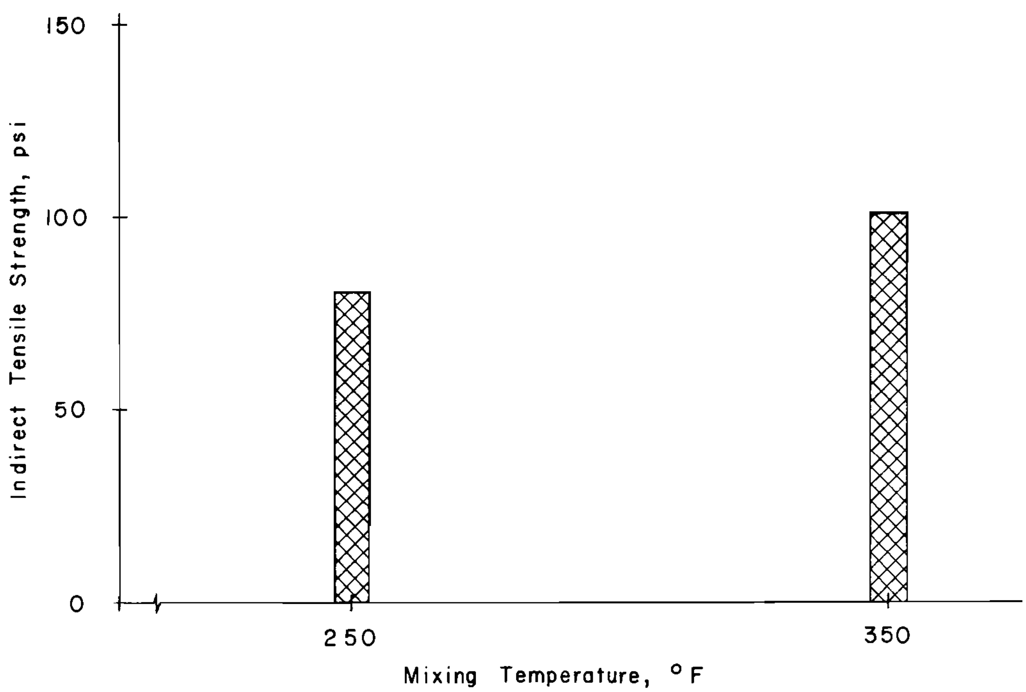

Fig 33. Effect of mixing temperature on tensile strength (Factor F).

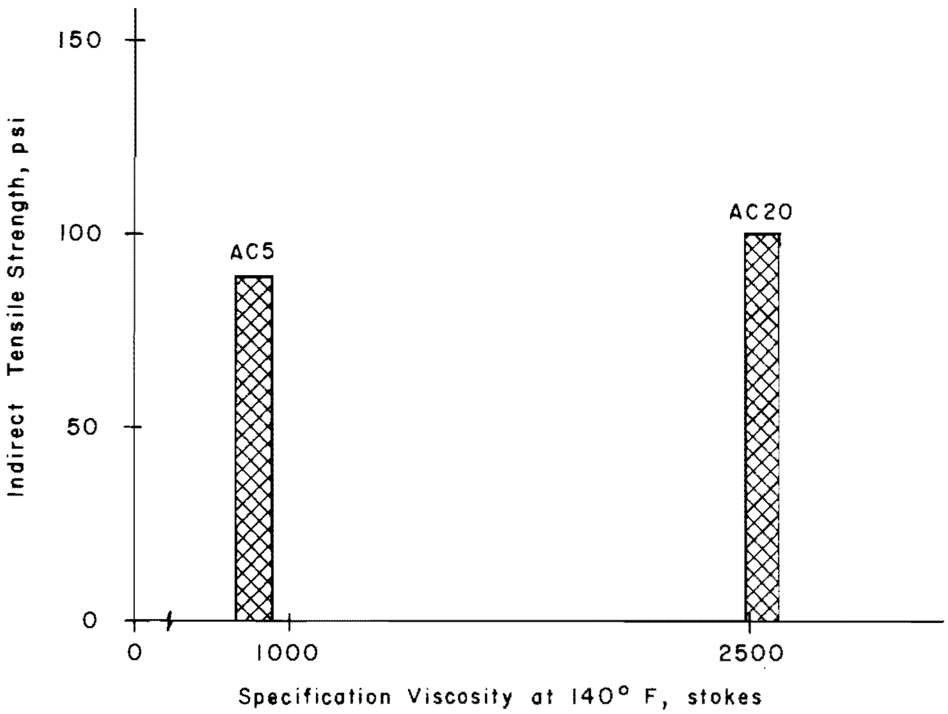

Fig 34. Effect of asphalt viscosity on tensile strength (Factor C).

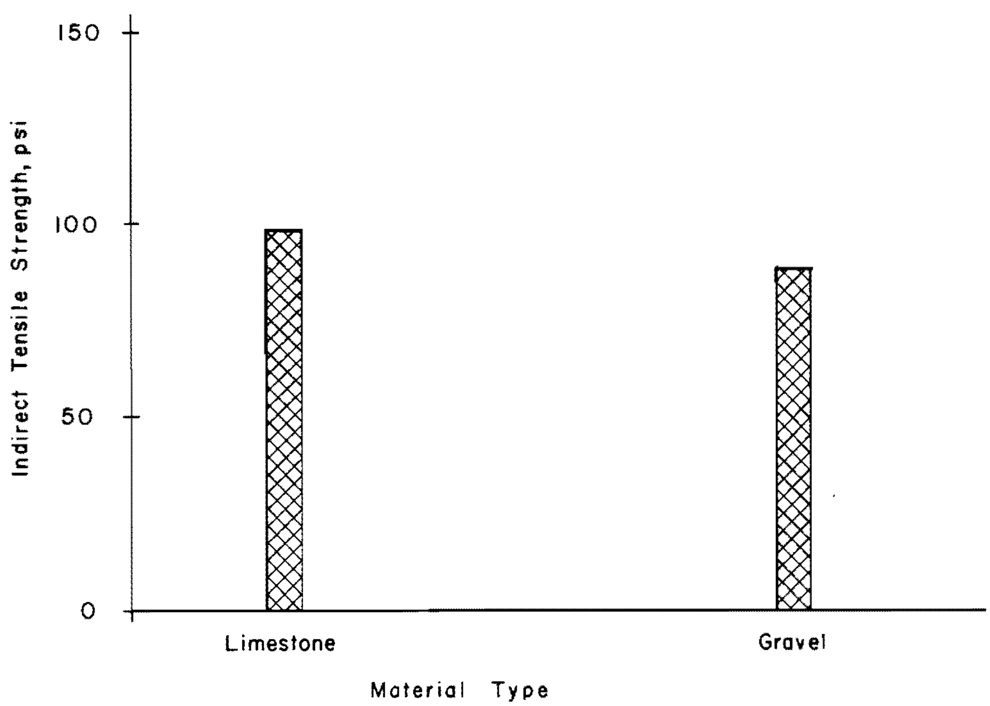

Fig 35. Effect of aggregate type on tensile strength (Factor A).

(3) using impact rather than gyratory-shear compaction (Fig 32),
(4) increasing mixing temperature from 250^{o} to 350^{o} F (Fig 33),
(5) using an AC-20 rather than AC-5 asphalt cement (Fig 34), and
(6) using crushed limestone rather than rounded gravel aggregate (Fig 35).

HORIZONTAL FAILURE DEFORMATION

The analysis of variance for this parameter is presented in Table 11. In contrast to the results for the tensile strength, there were only eight main effects and interactions significant at a level of 0.01.

Three-Way Interactions

There were only two three-way interactions significant at the 0.05 level or greater with both of them significant at the 0.01 alpha level. The significant interactions involved aggregate gradation, asphalt content, and curing temperature and aggregate gradation, compaction type, and compaction temperature.

Aggregate Gradation × Asphalt Content × Curing Temperature (Interaction B×D×H). The three-way interaction between the asphalt content, gradation type, and curing temperature influenced significantly the results obtained for specimens tested in tension as seen in Fig 36. The three-dimensional plot shows the three-way interaction associated with each gradation type. The coarse gradation in conjunction with the other two factors generally produced specimens with greater values of horizontal failure deformation.

Aggregate Gradation × Compaction Type × Compaction Temperature (Interaction B×E×G). Another significant three-way interaction, shown in Fig 37, involved gradation type, compaction type, and compaction temperature. From the plot it is apparent that the mixtures containing coarse gradations generally exhibited greater horizontal deformation values.

Two-Way Interaction

The only two-way interaction which was significant at the 0.01 alpha level involved aggregate type and asphalt content (Interaction A×D). The effect of the two-way interaction of asphalt content and aggregate type on the horizontal deformation is depicted in Fig 38. There was a slight increase

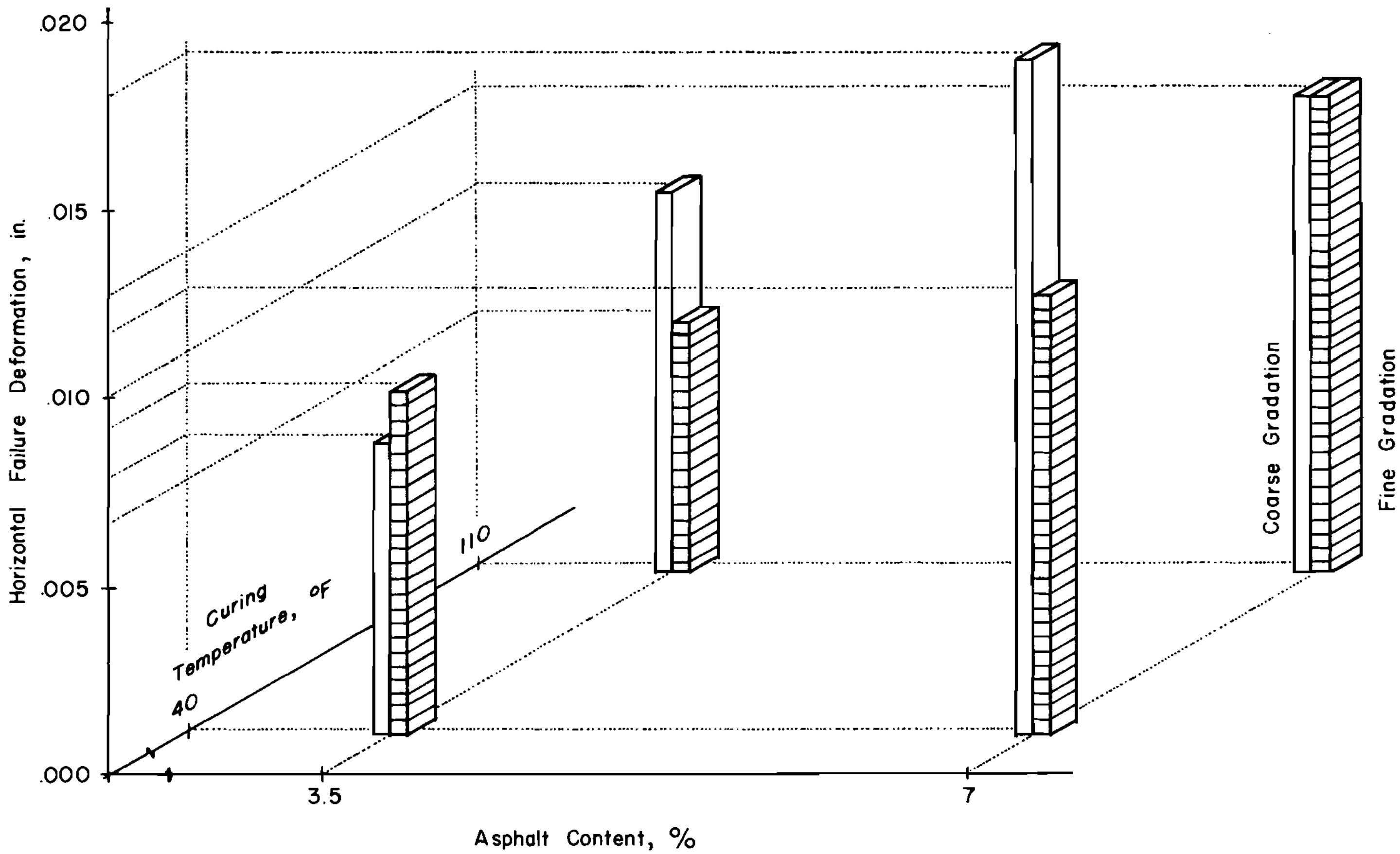

Fig 36. Effect of interaction between aggregate gradation, asphalt content, and curing temperature on horizontal failure deformation (Interaction B×D×H).

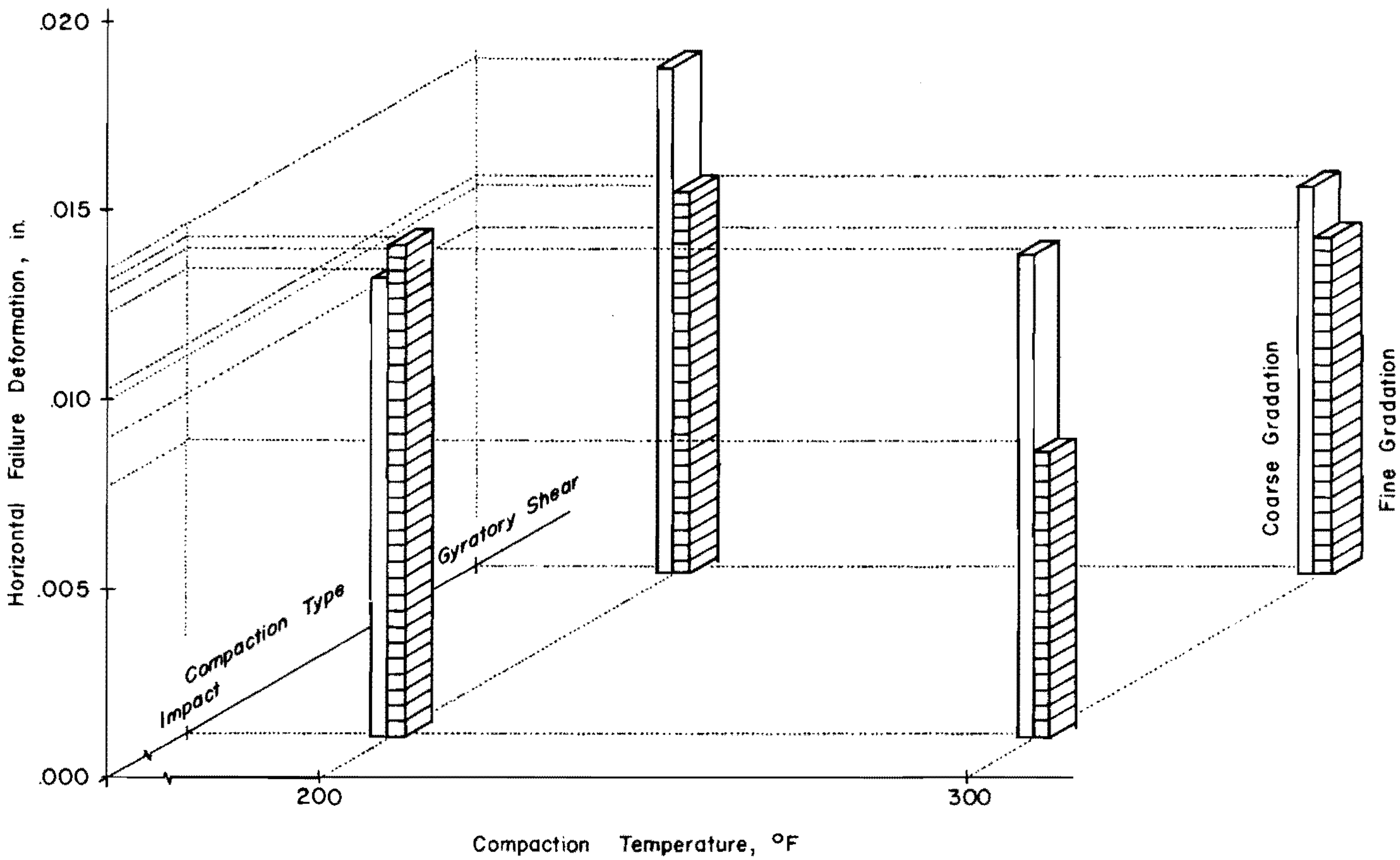

Fig 37. Effect of interaction between aggregate gradation, compaction type, and compaction temperature on horizontal failure deformation (Interaction B×E×G).

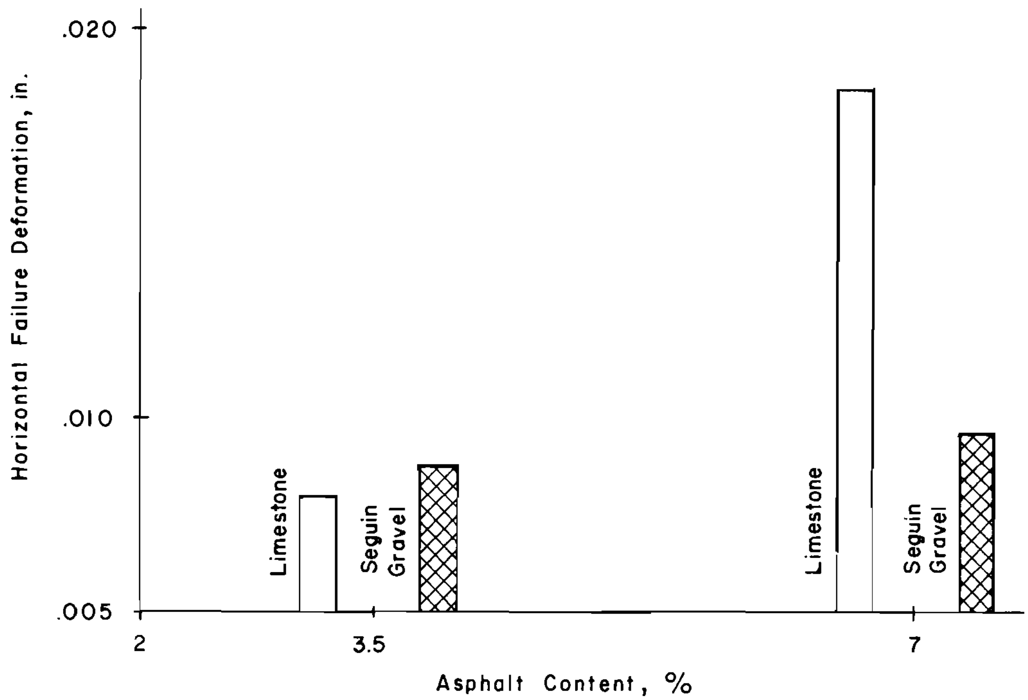

Fig 38. Effect of interaction between aggregate type and asphalt content on horizontal failure deformation (Interaction A×D).

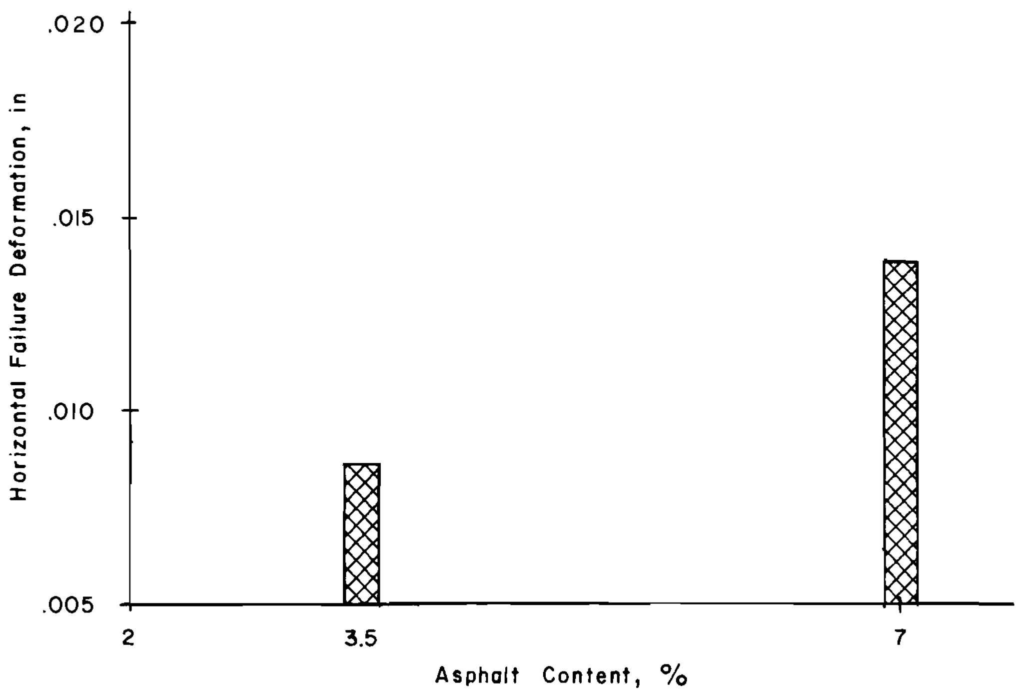

Fig 39. Effect of asphalt content on horizontal failure deformation (Factor D).

in the deformation value associated with the increase in asphalt content for those specimens containing Seguin gravel.

Main Effects

The factors significant at the 0.01 level included asphalt content, mixing temperature, aggregate type, compaction temperature, and gradation. The effects of these factors are presented in Figs 39-43. From these figures it can be seen that the horizontal failure deformation was increased significantly by

(1) increasing the asphalt content from 3.5 percent to 7.0 percent (Fig 39),

(2) decreasing the mixing temperature from 350° to 250° F (Fig 40),

(3) using crushed limestone rather than rounded Seguin gravel (Fig 41),

(4) decreasing the compaction temperature from 300° to 200° F (Fig 42), and

(5) using a coarse gradation rather than a fine gradation of aggregate (Fig 43).

Regression Equations

A regression analysis was conducted for each dependent variable to obtain a prediction equation based upon those factors and interactions significant at an alpha level of 0.05. From these equations a prediction of the tensile strength or horizontal failure deformation within some standard error can be made for combinations of the independent variables. The equations are as follows:

(1) Tensile Strength S_T , psi,

$$
\begin{aligned}
S_T = {} & 186.548 - 27.386D_i - 0.508G_i + 0.345B_iD_i \\
& - 8.277A_iD_i + 0.107D_iF_i - 1.810E_i - 0.588F_i \\
& + 0.004C_iD_i - 3.771D_iE_i - 0.031C_i + 0.130D_iG_i \\
& + 22.181A_i + 0.002B_iC_i + .044D_iH_i + 0.104\ E_iG_i
\end{aligned}
$$

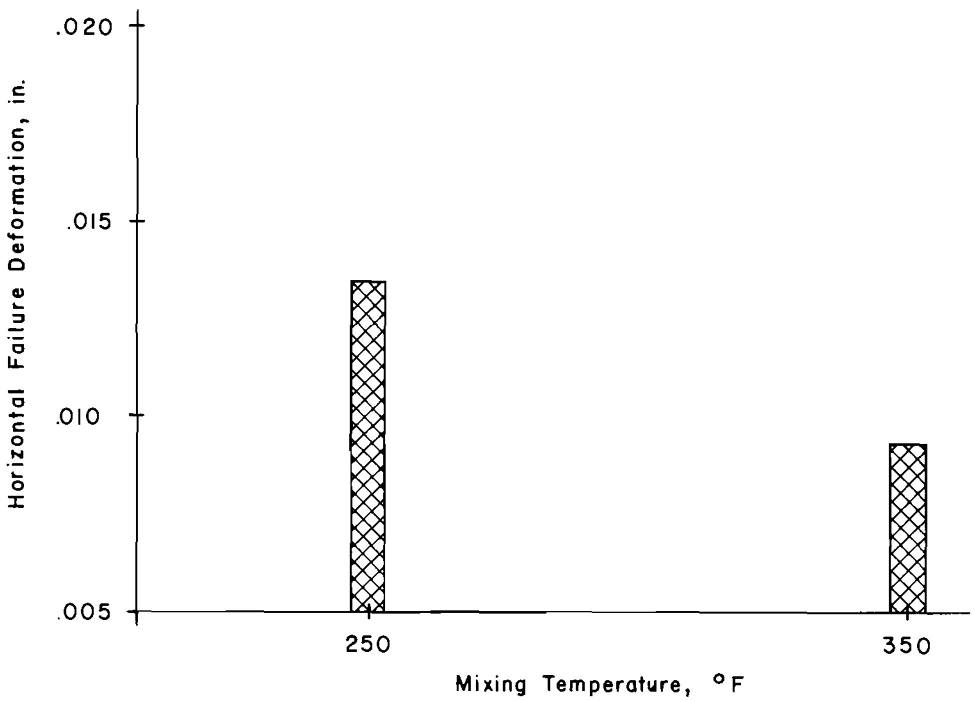

Fig 40. Effect of mixing temperature on horizontal failure deformation (Factor F).

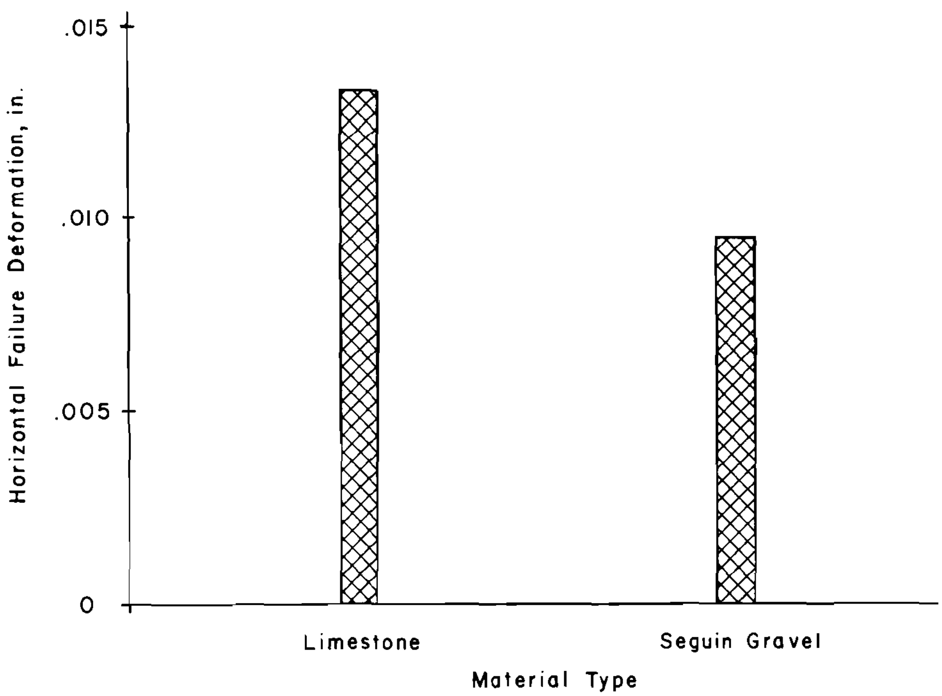

Fig 41. Effect of aggregate type on horizontal failure deformation (Factor A).

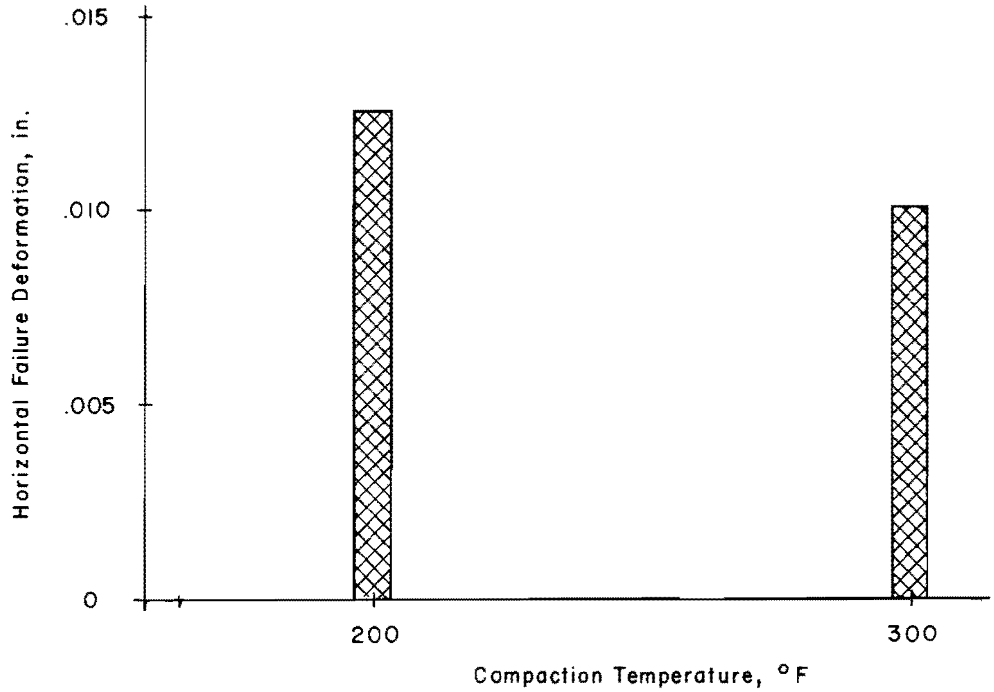

Fig 42. Effect of compaction temperature on horizontal failure deformation (Factor G).

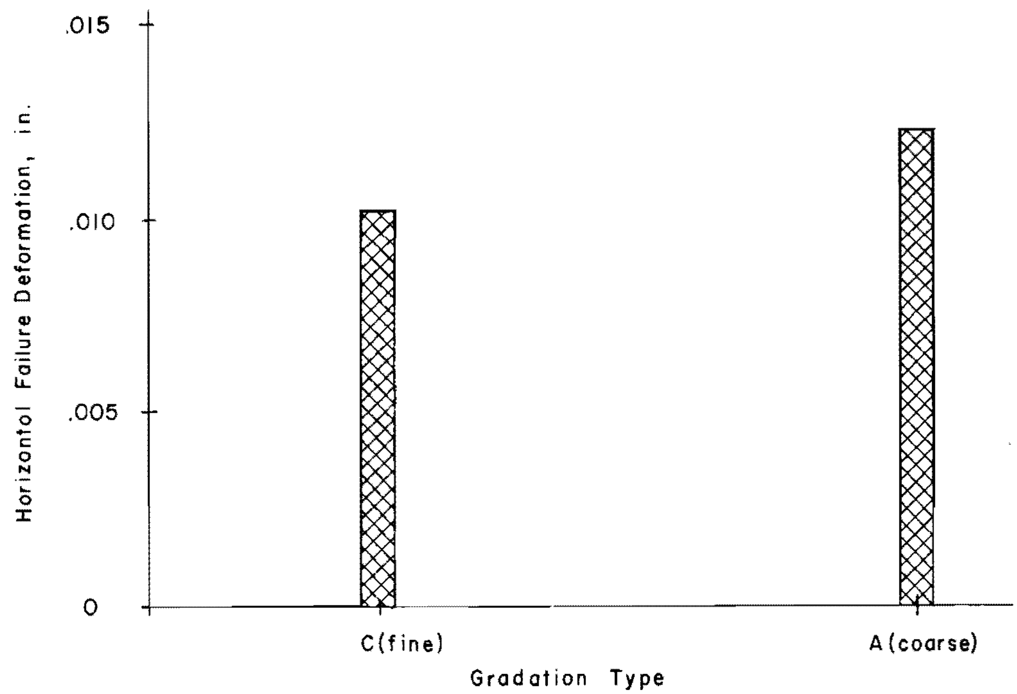

Fig 43. Effect of aggregate gradation on horizontal failure deformation (Factor B).

$$- 0.146A_iH_i - 0.013B_iD_iG_i + 0.004A_iC_i - .002B_iE_iG_i$$

$$+ 3.252A_iE_i - 1.036A_iB_i + 0.061A_iF_i + 0.060A_iG_i$$

$$+ 6.417B_i - 0.034B_iF_i \quad (9)$$

where S_T = predicted tensile strength and A_i, B_i, C_i, D_i, E_i, F_i, G_i, and H_i = levels of the eight independent variables (see Table 12 for levels used in the experiment.

The standard error of estimate was equal to ± 17.37 while the multiple correlation coefficient for the equation was 0.9793.

(2) Horizontal Failure Deformation D_h, inches,

$$D_h = + .00442 + .00425D_i + 0.00140A_iD_i + 0.00001$$

$$(2F_i + G_i - D_iG_i - D_iF_i + 2A_iH_i + H_i - A_iG_i - A_iF_i$$

$$- E_iF_i) - 0.00066A_i - 0.00128B_i + 0.00019B_iD_i \quad (10)$$

where D_h = horizontal failure deformation and A_i, B_i, D_i, F_i, G_i, and H_i = levels of the independent variables (see Table 12 for levels used in experiment).

The standard error of estimate for the equation above was ± 0.00560. The multiple correlation coefficient was 0.9182.

DISCUSSION OF POSTULATIONS

This experiment was not designed to explain the observed effects produced by all the factors. It is desirable and possible, however, to compare these results with several theories which have been discussed in other literature and to formulate a reasonable hypothesis of tensile behavior. These postulations will help in planning future work.

TABLE 12. LEVELS OF FACTORS USED IN REGRESSION EQUATIONS

Factor	Description	Level
Aggregate type	Limestone	$A_1 = 2$
	Seguin gravel	$A_2 = 0$
Aggregate gradation	Fine	$B_1 = 2$
	Coarse	$B_2 = 8$
Asphalt viscosity	AC-5	$C_1 = 773$
	AC-20	$C_2 = 2532$
Asphalt content	Low	$D_1 = 3.5$
	High	$D_2 = 7.0$
Compaction type	Impact	$E_1 = 2$
	Gyratory	$E_2 = 0$
Mixing temperature	Low	$F_1 = 250$
	High	$F_2 = 350$
Compaction temperature	Low	$G_1 = 200$
	High	$G_2 = 300$
Curing temperature	Low	$H_1 = 40$
	High	$H_2 = 110$

Asphalt Content (Factor D)

The asphalt content had the greatest effect on both the tensile strength and horizontal deformation of the asphalt-treated specimens. At the high asphalt content specimens exhibited greater tensile strengths and deformation values. This can probably be explained by the manner in which the asphalt combines with the aggregate particles. There are two concepts which explain this phenomena (Ref 90). The first of these is the "intimate mixture," in which an asphalt film is supposed to surround each individual soil particle. The strength of such a mixture is dependent primarily upon the viscosity of the asphalt and the adhesion between soil particles and asphalt. The second concept is referred to as the "plug" theory. The asphalt in this mixture does not coat each individual particle but does coat conglomerate particles. The strength of this system is governed by the viscosity of the asphalt and the degree of adhesion between the aggregate particles.

At the lower asphalt contents the second concept probably explains the combination of the asphalt binder and aggregate. It is probable that the amount of asphalt present was insufficient to provide a complete asphalt-aggregate matrix. Since the tensile strength of such a specimen was primarily provided by the asphalt binder, the resulting strength was lowered.

With increased amounts of asphalt, the mixture more closely resembled an intimate mix since the asphalt films generally surrounded and coated more of the aggregate particles. This resulted in a corresponding increase in tensile strength.

Compaction and Mixing Temperatures

The effects of compaction and mixing temperatures were very similar, with the high level of each associated with greater tensile strengths and smaller horizontal deformations. The increased temperatures decreased the viscosity of asphalt, thereby allowing it to flow more easily over the surface of the aggregate. At low asphalt contents the decreased viscosity or increased wetting ability was hampered by an insufficient amount of asphalt to coat all particles. At high asphalt contents, the increased temperatures helped to produce "intimate" mixtures with thin asphalt films binding the aggregate particles together. When subjected to tensile stresses, the matrix of aggregate and asphalt withstood greater stresses with little or no flow of the asphalt; therefore, smaller deformation occurred at failure. At the lower temperatures with high asphalt contents, less asphalt was absorbed by the aggregate, resulting in thicker asphalt films binding the aggregate particles together. When tested in tension, the thicker films allowed more movement or deformation in the asphalt under the tensile stresses until the asphalt film failed or the stresses were transferred to the aggregate particles.

Aggregate Type

For the two types of aggregate evaluated, the asphalt-treated mixtures with limestone exhibited greater average tensile strengths and also greater horizontal failure deformations than those containing Seguin gravel. The major physical differences between the two types were the porosity and surface texture. Seguin gravel is a relatively smooth textured nonporous aggregate while the crushed limestone is composed of rough textured angular particles with high porosity. The greater porosity, angularity, and rougher texture increased the adhesion between the aggregate particles and asphalt, producing a mixture which could withstand greater horizontal deformations.

At an asphalt content of 3.5 percent the amount of residual asphalt, i.e., that not absorbed by the aggregate, was only sufficient to coat conglomerate particles forming a relatively weak asphalt-aggregate matrix. The limestone, due to its high degree of porosity, absorbed a greater percentage of the available asphalt than the Seguin gravel to form mixtures with greater adhesion. Therefore, the mixtures containing limestone aggregate exhibited greater tensile strengths.

At 7.0 percent content sufficient asphalt was available with both aggregate types to form mixtures which closely approximate "intimate mixes." The adhesive bond between the aggregate and asphalt for both cases was then very strong. The tensile strength in these cases probably depended upon the strength of aggregate as well as adhesive bond. In these specimens the tensile strength of limestone aggregate was felt to be less than the adhesive strength of the aggregate-asphalt matrix. On the other hand the tensile strength of Seguin gravel is greater than limestone and, therefore, the strength of Seguin gravel specimens was greater than limestone specimens.

Aggregate Gradation and Asphalt Viscosity

Use of coarse aggregate gradations produced specimens which exhibited large horizontal deformations before failure. This can probably be explained primarily by the amount of voids and residual asphalt present in the mix.

For a fixed asphalt content a mixture with a fine aggregate gradation, because of the relatively large surface to volume ratio of the small particles, absorbs more of the asphalt, leaving a smaller amount of residual asphalt. Fine gradations also insure fewer voids in the mixture because during compaction the mastic formed by the combination of fine particles with the asphalt fills a larger portion of the voids between the larger particles. This phenomenon does not occur for mixtures composed of coarse aggregate gradations and asphalt, and thus they exhibit more voids and more residual asphalt. When tested in tension, the combination of these two parameters allows more asphalt flow and more reorientation of aggregate particles within the mixture before failure, therefore producing greater horizontal deformations.

The analysis of the interaction between asphalt viscosity and aggregate gradation showed that the change in viscosity had a greater effect on the mixtures containing coarse gradations. In fact the viscosity change had essentially no effect on the tensile strength of fine graded asphalt-treated mixtures. This interaction effect on tensile strengths may be explained by the following theory.

During the mixing phase the asphalt in such mixes is more readily absorbed by the fine particles to form a mastic. During the compaction phase this mastic fills the voids between the larger particles and cements them together into a denser mix. This theory is very similar to the mastic theory proposed by Csanyi (Refs 24 and 91) and Oberbach (Ref 92). The strength of such a mixture is dependent more upon the adhesion obtained between mastic and larger particles than on the viscosity of the asphalt used. On the other hand the coarse gradation produces significantly stronger specimens when combined with the more viscous asphalt. The coarse gradation has significantly fewer fines; therefore, the amount of mastic formed is relatively small. The mixture formed is less dense and is held together primarily by the asphalt films between the aggregate particles. The strength of mixtures composed of this coarse gradation is dependent more upon the viscosity or resistance to flow of the asphalt films than the adhesion provided by the small amount of mastic present in the material.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

This investigation was performed to evaluate the effects of eight factors and their interactions on the tensile properties of asphalt-treated materials. Two levels of each factor were established as the upper and lower bounds for this study. The results are then applicable only within the confines of the limits established for the eight factors.

The conclusions drawn from this experiment are applicable only within the inference space or population defined by the two levels of each main effect. There is no attempt made to apply the results outside of this particular inference space nor to evaluate nonlinear effects. The latter is not meant to imply that nonlinearity does not exist. In fact in many cases it is probable that the effects are in reality curvilinear. In future experiments this peaking or nonlinear behavior will be analyzed for a more thorough understanding of the factors affecting the tensile properties of asphalt-treated materials.

CONCLUSIONS

From this analysis the following conclusions were drawn.

(1) There were six main effects, 12 two-way interactions, and three three-way interactions which had a highly significant effect on the tensile strength of asphalt-treated materials and were considered of practical significance to the engineer. Included among the significant effects were

<u>Main Effects</u>:

(a) aggregate type,

(b) specification viscosity (AC-5 vs AC-20),

(c) asphalt content,

(d) compaction type,

(e) mixing temperature, and

(f) compaction temperature;

<u>Two-Way Interactions</u>:

(a) aggregate type and specification viscosity,

(b) aggregate type and asphalt content,

(c) aggregate type and compaction type,

(d) aggregate type and curing temperature,

(e) gradation and specification viscosity,

(f) gradation and asphalt content,

(g) specification viscosity and asphalt content,

(h) asphalt content and compaction type,

(i) asphalt content and mixing temperature,

(j) asphalt content and compaction temperature,

(k) asphalt content and curing temperature, and

(l) compaction type and compaction temperature;

Three-Way Interactions:

(a) gradation, asphalt content, and compaction temperature,

(b) gradation, compaction type, and compaction temperature, and

(c) specification viscosity, compaction type, and mixing temperature.

(2) There were five main effects, one two-way interaction, and two three-way interactions which had highly significant effects ($\alpha = 0.01$) on the horizontal deformation of asphalt-treated materials. Included among the significant effects were

Main Effects:

(a) aggregate type,

(b) gradation,

(c) asphalt content,

(d) mixing temperature, and

(e) compaction temperature;

Two-Way Interaction:

aggregate type and asphalt content;

Three-Way Interactions:

(a) gradation, asphalt content, curing temperature, and

(b) gradation, compaction type, and compaction temperature.

(3) The existence of the large number of highly significant main effects and interactions illustrates the complexity of the relationship between the tensile properties of asphalt-treated materials and a number of the independent factors.

(4) In general it was found that tensile strength was increased by

(a) increasing the asphalt content from 3.5 percent to 7.0 percent,

(b) increasing the compaction temperature from 200° F to 300° F,

(c) using impact rather than gyratory-shear compaction,

(d) increasing the mixing temperature from 250° F to 350° F,

(e) using an AC-20 rather than an AC-5, and

(f) using crushed limestone rather than rounded gravel aggregate.

(5) In general, it was found the horizontal failure deformation was increased by

(a) increasing the asphalt content from 3.5 percent to 7.0 percent,

(b) decreasing the mixing temperature from 350° F to 250° F,

(c) using crushed limestone rather than rounded Seguin gravel,

(d) decreasing the compaction temperature from 300° F to 200° F, and

(e) using the coarse gradation rather than the fine gradation.

(6) Since there are several two-way and three-way interactions between main factors which were important in establishing tensile strength and horizontal failure deformation, it is not adequate to infer only to a specific combination of factors based on main effects because consideration must be given to any interaction effects in predicting a dependent variable.

(7) Within the limits of this study, asphalt content appeared to have the greatest effect on the tensile strength of an asphalt-treated material. This was evidenced by the fact that the main effect of asphalt content, all two-way interactions and two three-way interactions involving asphalt content had highly significant effects on the tensile strength.

RECOMMENDATIONS

The following recommendations are made for further research into the factors affecting the tensile properties of asphalt-treated materials:

(1) A theoretical development relating the elastic properties of materials, i.e., Poisson's ratio and modulus of elasticity, to the applied load and corresponding total vertical and horizontal strains obtained in the indirect tensile test would be very beneficial in the evaluation of the factors affecting the tensile properties of asphalt-treated materials.

(2) A detailed look at the effect of several different asphalt contents on the tensile properties of stabilized mixtures could provide sufficient information to develop adequate predictive

equations for each dependent variable. At the same time intermediate values of the other quantitative independent variables, e.g., compaction temperature and mixing temperature, could be entered into the study. Optimization techniques could then be utilized on the data obtained from such an expanded study to estimate the value of each independent variable, which should be specified to obtain the optimum value for the dependent variable, i.e., tensile strength, modulus of elasticity, or Poisson's ratio.

(3) An investigation of the effect of the significant factors on the tensile properties of asphalt-treated materials in repeated loading should be undertaken in future testing.

(4) Consideration should be given in future experiments to an evaluation of the possible effect of phasing, i.e., mixing, compaction, and curing, on the experimental error. The results from such a study would determine the type of analysis required for that particular experiment and might cause a change in the order and significance of the factors and their interactions.

APPLICATION OF RESULTS

This report describes work which is a building block for further developments in this project. It was not intended for direct application in the field, but as a screening experiment for the project. However, findings reported herein can be used to insure that complex materials such as asphaltic material are studied adequately before conclusions are drawn based on main effects alone. The work may lead to an improved method of asphaltic mix design by providing realistic ways for evaluating tensile properties.

In addition, the test results begin to establish the range of tensile strengths which can be expected from asphalt-treated contents and the factors which affect these strengths. Although these strength values are only approximate, they should be of immediate value to engineers concerned with pavement design or the development of pavement design and evaluation techniques.

REFERENCES

1. Hargett, E. R., "Basic Material Properties for the Design of Bituminous Concrete Surfaces," International Conference on the Structural Design of Asphalt Pavements, University of Michigan, 1962, pp 326-384.

2. Peattie, K. R., "A Fundamental Approach to the Design of Flexible Pavements," International Conference on Structural Design of Asphalt Pavements, University of Michigan, 1962, pp 487-494.

3. Westergaard, H. M., "Analytical Tools for Judging Results of Structural Tests of Concrete Pavements," Public Roads, Vol 14, 1938.

4. Westergaard, H. M., "Computation of Stresses in Concrete Roads," Proceedings, Fifth Annual Meeting of the Highway Research Board, 1926.

5. Hudson, W. Ronald, and Hudson Matlock, "Discontinuous Orthotropic Plates and Pavement Slabs," Research Report No. 56-6, Center for Highway Research, The University of Texas, Austin, May 1966.

6. Whiffin, A. C., and W. W. Lister, "The Application of Elastic Theory to Flexible Pavements," International Conference on Structural Design of Asphalt Pavements, University of Michigan, 1962, pp 611-634.

7. Hudson, W. Ronald, and T. W. Kennedy, "An Indirect Tensile Test for Stabilized Materials," Research Report No. 98-1, Center for Highway Research, The University of Texas at Austin, January 1969.

8. Stevens, D. E., "Fundamentals of Stability Testing of Asphalt Mixes," Proceedings of the Association of Asphalt Paving Technologists, Vol 22, 1953, pp 364-382.

9. Soil Mechanics for Road Engineers, Stabilization of Soil with Bituminous Materials, Her Majesty's Stationery Office, 1957, pp 253-272

10. Neppe, S. L., "Mechanical Stability of Bituminous Mixtures: A Summary of the Literature," Proceedings of the Association of Asphalt Paving Technologists, Vol 22, 1953, pp 383-417.

11. Anderson, K. O., R. C. G. Haas, and A. D. Laplante, "Triaxial Shear Strength Characteristics of Some Sand-Asphalt Mixtures," Highway Research Record, No. 91, Highway Research Board, pp 1-12.

12. McLeod, N. W., "The Rational Design of Bituminous Paving Mixtures," Proceedings, Vol 29, Highway Research Board, 1949.

13. McLeod, N. W., "A Rational Approach to the Design of Bituminous Paving Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 19, 1950.

14. Messina, R., "Split Cylinder Test for Evaluation of the Tensile Strength of Asphalt Concrete Mixtures," Master's Thesis, The University of Texas, 1966.

15. Akazawa, T., "Tension Test Method for Concrete," International Association of Testing and Research Laboratories for Materials and Structures, Paris, Bulletin No. 16, November 1953.

16. Carneiro, F. L. L. B., and A. Barcellos, "Concrete Tensile Strength," International Association of Testing and Research Laboratories for Materials and Structures, Paris, Bulletin No. 13, March 1953.

17. Frocht, M. M., "Photoelasticity," Vol 2, John Wiley and Sons, Inc., 1957.

18. Foppl, A., and L. Foppl, Drang and Zwang, Vol I, Oldenburg Verlag, Berlin, 1941.

19. Peltier, R., "Theoretical Investigation of the Brazilian Test," International Association of Testing and Research Laboratories for Materials and Structures, Paris, Bulletin No. 19, March 1953, pp 97-123.

20. Ramesh, C. K., and S. K. Chopra, "Determination of Tensile Strength of Concrete and Mortar by the Split Test," Indian Concrete Journal, September 1960, pp 254-257.

21. Timoshenko, S., and J. N. Goodier, Theory of Elasticity, 2nd Edition, McGraw-Hill, New York, 1951.

22. Hertz, H., "Über die Verteilung der Druckräfte in einem elastichen Kreiscylinder," Zeitschrift für Mathematik und Physik, Vol 28, 1883.

23. Hondros, G., "The Evaluation of Poisson's Ratio and the Modulus of Materials of a Low Tensile Resistance by the Brazilian (Indirect Tensile) Test with Particular Reference to Concrete," Australian Journal of Applied Science, Vol 10, No. 3.

24. Csanyi, L. H., and H. P. Fung, "Mortar Theory for Use of Ungraded Aggregates in Bituminous Mixes," Bulletin, No. 109, Highway Research Board, January 1955.

25. Monismith, C. L., "Asphalt Paving Mixtures Properties, Design, and Performance," Course Notes in Asphalt Paving prepared by Institute of Transportation and Traffic Engineers, University of California, 1961, pp 62-65.

26. Stanton, T. E., and F. N. Hveem, "Role of the Laboratory in the Preliminary Investigation and Control of Materials for Low Cost Bituminous Pavements," Proceedings, Vol 14, Part II, Highway Research Board, 1935.

27. Hveem, F. N., "Gradation of Mineral Aggregates for Dense Graded Bituminous Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 11, 1940, p 315.

28. Griffith, J. M., and B. F. Dallas, "Influence of Fine Aggregates on Asphaltic Concrete Paving Mixtures," Proceedings, Vol 37, Highway Research Board, 1958, pp 219-255.

29. Wedding, P. A., and R. D. Gaynor, "The Effects of Using Crushed Gravel as the Coarse and Fine Aggregate in Dense Graded Bituminous Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 30, 1961, pp 469-492.

30. Campen, W. H., and J. R. Smith, "A Study of the Role of Angular Aggregates in the Development of Stability in Bituminous Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 17, 1948, pp 114-142.

31. Smith, F. P., "Essential Physical Properties of Sand, Gravel, Slags, and Broken Stone for Use in Bituminous Pavements," Better Roads and Streets, Vol 6, No. 3, 1916, p 18.

32. Monismith, C. L., "Fatigue of Asphalt Paving Mixtures," A paper presented at the First Annual Street and Highway Conference, University of Nevada, March 9, 1966.

33. Proudley, C. E., and H. F. Waller, Jr., "Some Considerations in the Use of Gravel for Bituminous Paving Mixtures in North Carolina," Proceedings of the Association of Asphalt Paving Technologists, Vol 26, 1957, pp 23-37.

34. Neppe, S. L., "Mechanical Stability of Bituminous Mixtures: A Summary of the Literature," Proceedings of the Association of Asphalt Paving Technologists, Vol 22, 1955, pp 383-427.

35. Hubbard, P., "Effect of Aggregates Upon Stability of Bituminous Mixtures," Proceedings, Vol 29, American Society for Testing Materials, 1929, p 803.

36. Vokac, R., "A Type of Aggregate Gradation for Non-critical Asphaltic Paving Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 8, 1939, p 86.

37. Spielmann, P. E., and A. C. Hughes, "Asphalt Roads," Roadmakers' Library, Vol 5, 1936.

38. Olmstead, F. R., "Avoiding Trouble in the Design and Control of Oil Aggregate Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 11, 1940, p 258.

39. Pell, P. S., "Fatigue of Asphalt Pavement Mixes," Proceedings, Second International Conference on the Structural Design of Asphalt Pavement, University of Michigan, 1967.

40. Herrin, M., Highway Engineering Handbook, Section 21, pp 21-85.

41. Hubbard, P., and F. C. Field, "The Rational Design of Asphalt Paving Mixtures," Research Series 1, Asphalt Institute, 1935.

42. Pauls, J. T., and J. F. Goode, "Application and Present Status of the Immersion-Compression Test," Proceedings of the Association of Asphalt Paving Technologists, Vol 16, 1947.

43. Vallerga, B. A., "Recent Laboratory Compaction Studies of Bituminous Paving Mixtures," Proceedings of the Association of Asphalt Paving Technologists, Vol 20, 1951.

44. Phillippi, O. A., "Molding Specimens of Bituminous Paving Mixtures," Proceedings, Vol 31, Highway Research Board, 1952.

45. Mix Design Methods for Asphalt Concrete, The Asphalt Institute, Manual Series (MS-2), pp 67-68.

46. McRae, J. L., "Compaction of Bituminous Concrete," Proceedings of the Association of Asphalt Paving Technologists, Vol 26, 1957, pp 206-213.

47. Johnson, A. W., and J. R. Sallberg, "Factors Influencing Compacting Test Results," Bulletin, No. 319, Highway Research Board, 1962.

48. U. S. Corp of Engineers, "The California Bearing Ratio as Applied to the Design of Flexible Pavements for Airports," U. S. Waterways Experiment Station, Technical Memo 213-1, July 1945.

49. McLeod, N. W., "Influence of Viscosity of Asphalt-Cements on Compaction of Paving Mixtures in the Field," Highway Research Record, No. 158, Highway Research Board, 1967, pp 76-115.

50. Nevitt, H. G., "Asphalt Pavement Construction Temperatures," Roads and Streets, Vol 102, January 1959, pp 176-177.

51. Griffith, J. M., "How Viscosity at Mixing Temperatures Affects the Mix," Roads and Streets, Vol 102, June 1959, pp 162-165.

52. Rice, J. M., "Effects of Temperature on Properties of Bituminous Mixtures," Special Report, No. 54, Highway Research Board, 1960, pp 2-4.

53. Shoor, S.K., K. Majidzadeth, and H. E. Schweyer, "Temperature Flow Functions for Certain Asphalt Cements," Highway Research Record, No. 134, Highway Research Board, 1966, pp 63-74.

54. Adams, V., "The Effects of Air and Base Temperature on Compaction," Special Report, No. 54, Highway Research Board, pp 36-67.

55. Hveem, F. N., "Effects of Time and Temperature on Hardening of Asphalts," Special Report, No. 54, Highway Research Board.

56. Herrin, M., Highway Engineering Handbook, McGraw-Hill, Inc., 1960, Section 21, pp 21-85.

57. Mack, C., "A Quantitative Approach to the Measurement of the Bearing Strength of Road Surfaces," Proceedings of the Association of Asphalt Paving Technologists, Vol 17, 1947.

58. Neppe, S. L., "The Influence of Rheological Characteristics of the Binder on Certain Mechanical Properties of Bitumen-Aggregate Mixes," Proceedings of the Association of Asphalt Paving Technologists, Vol 22, 1952, pp 428-473.

59. Heukelom, W., and A. J. G. Klomp, "Road Design and Dynamic Loading," Paper to be presented before the Annual Meeting of the Association of the Asphalt Paving Technologists to be held in Dallas from 17-19 February 1964.

60. Pell, P. S., "Fatigue Characteristics of Bitumen and Bituminous Mixes," International Conference on the Structural Design of Asphalt Pavements, University of Michigan, 1963, pp 280, 282, 294-297, 310-323.

61. Breen, J. J., and J. E. Stephens, "Split Cylinder Test Applied to Bituminous Mixtures at Low Temperatures," Journal of Materials, Vol 1, No. 1, American Society for Testing and Materials, March 1966.

62. Heukelom, W., and A. J. G. Klomp, "Road Design and Dynamic Loading," Proceedings of the Association of Asphalt and Paving Technologists, Vol 24, 1964.

63. Hughes, E. C., and R. B. Farris, "Low Temperature Maximum Deformability of Asphalts," Proceedings of the Association of Asphalt Paving Technologists, Vol 19, 1950.

64. Nijboer, L. W., "The Determination of the Plastic Properties of Bitumen-Aggregate Mixtures and the Influence of Variations in the Composition of the Mix," Proceedings of the Association of Asphalt Paving Technologists, Vol 16, 1947.

65. Abdel-Hady, M., and M. Herrin, "Rheological Properties of Compacted Soil-Asphalt Mixtures," Highway Research Record, No. 91, Highway Research Board, pp 13-35.

66. Nijboer, L. W., "Mechanical Properties of Asphalt Materials and Structural Design of Asphalt Roads, Proceedings, Vol 33, Highway Research Board, 1954.

67. Wood, L. E., and W. H. Goetz, "Strength of Bituminous Mixtures and their Behavior under Repeated Loads," Proceedings, Vol 35, Highway Research Board, 1956.

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

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

70. Mack, C., "Rheology of Bituminous Mixtures Relative to the Properties of Asphalt," Proceedings of the Association of Asphalt Paving Technologists, Vol 13, 1942.

71. Muskhelishvili, N. I., "Some Basic Problems of the Mathematical Theory of Elasticity," P. Noordhoff Limited, pp 324-328.

72. Sokolnikoff, I. S., "Mathematical Theory of Elasticity," McGraw-Hill, New York, 1956, pp 28-284.

73. Wright, P. J. F., "Comments on an Indirect Tensile Test on Concrete Cylinders," Magazine of Concrete Research, Vol 17, No. 20, London, July 1955, pp 87-96.

74. Winterkorn, H. F., "Granulometric and Volumetric Factors in Bituminous Soil Stabilization," Proceedings, Vol 36, Highway Research Board, 1957.

75. Standard Specifications for Road and Bridge Construction, Texas Highway Department, 1962.

76. Welborn, J. Y., "Temperature-Viscosity Relation of Asphalts used in the United States," Special Report, No. 54, Highway Research Board, 1960, pp 5-6.

77. Rader, L. F., "Bituminous Paving Mixtures: Fundamentals for Design," Bulletin, No. 105, Highway Research Board, 1955.

78. "How Asphalt Grade and Filler Type Affect Pavement Compaction," Roads and Streets, Vol 103, No. 8, August 1960, pp 125-130.

79. Lefebvre, J. A. A., Private communication to N. W. McLeod (Ref 55).

80. Nevitt, H. B., "Compaction Fundamentals," Proceedings of the Association of Asphalt Paving Technologists, Vol 26, 1957, pp 201-206.

81. Seed, H. B., R. Lundgren, and C. K. Chan, "Effect of Compaction Method on Stability and Swell Pressure of Soils," Bulletin, No. 93, Highway Research Board, 1954, pp 33-48.

82. ASTM D1559-65, "Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus," ASTM Standards Part 11, Bituminous Materials: Soils Skid Resistance, March 1968, p 482.

83. Kimble, F. W., "Effect of Mix Temperature," Special Report, No. 54, Highway Research Board, pp 34-35.

84. Parker, C. F., "Effect of Mix Temperature," Special Report, No. 54, Highway Research Board, p 28.

85. Parker, C. F., "Steel Tired Rollers," Bulletin, No. 246, Highway Research Board, 1960, pp 1-40.

86. Nijboer, L. W., "Plasticity as a Factor in the Design of Dense Bituminous Road Carpets," Elsevier Publishing Company, Amsterdam, Netherlands.

87. Traxler, R. N., "Relation Between Asphalt Composition and Hardening by Volatilization and Oxidation," Proceedings of the Association of Asphalt Paving Technologists, Vol 30, 1961, pp 359-377.

88. Traxler, R. N., "Pavement Performance and Durability as Affected by Asphalt Properties," Proceedings of the Association of Asphalt Paving Technologists, Vol 32, 1963, pp 229-236.

89. Kempthorne, O., Design and Analysis of Experiments, John Wiley and Sons, Inc., 1952, Chapter 20, pp 390-409.

90. Endersby, V. A., "Fundamental Research in Bituminous Soil Stabilization," Proceedings, Vol 22, Highway Research Board, 1942, pp 442-459.

91. Csanyi, L. H., "Bituminous Mastic Surfacings," Proceedings of the Association of Asphalt Paving Technologists, Vol 25, 1956.

92. Oberbach, J., "Gussasphalt, Teer, und Asphalt Strassenbau," Strassenbau, Chemie und Technick Verlagsgesellschaft, Heidelberg, 1950, p 425.

APPENDIX 1

COMPARISON OF TEST GRADATIONS WITH STANDARD TEXAS HIGHWAY DEPARTMENT GRADATIONS

APPENDIX 1. COMPARISON OF TEST GRADATIONS WITH STANDARD TEXAS HIGHWAY DEPARTMENT GRADATIONS

Coarse Gradation

Test Gradation A

Texas Highway Department Type A (Coarse Graded Base Course)

	THD "A" SPECS, % by Wgt	TEST GRADATION A, % by Wgt	WITHIN THD SPECS
Passing 2" sieve	100	100	Yes
Passing 1-3/4" sieve	95-100	100	Yes
Passing 1-3/4" sieve, Retained on 7/8" sieve	15-40	0	No
Passing 7/8" sieve, Retained on 3/8" sieve	15-40	26	Yes
Passing 3/8" sieve, Retained on No. 4 sieve	10-25	21	Yes
Passing No. 4 sieve, Retained on No. 10 sieve	5-20	15	Yes
Total retained on No. 10 sieve	65-80	62	No
Passing No. 10 sieve, Retained on No. 40 sieve	0-20	15	Yes
Passing No. 40 sieve, Retained on No. 80 sieve	3-15	5	Yes
Passing No. 80 sieve, Retained on No. 200 sieve	2-15	8	Yes
Passing No. 200 sieve	0-8	10	No

Fine Gradation

Test Gradation C

Texas Highway Department Type D (Fine Graded Surface Course)

	THD "D" SPECS, % by Wgt	TEST GRADATION C, % by Wgt	WITHIN THD SPECS
Passing 1/2" sieve	100	98	No
Passing 3/8" sieve	95-100	95	Yes
Passing 3/8" sieve, Retained on No. 4 sieve	20-50	20	Yes
Passing No. 4 sieve, Retained on No. 10 sieve	10-30	15	Yes
Total Retained on No. 10 sieve	60-75	40	No
Passing No. 10 sieve, Retained on No. 40 sieve	0-30	30	Yes
Passing No. 40 sieve, Retained on No. 80 sieve	4-25	10	Yes
Passing No. 80 sieve, Retained on No. 200 sieve	3-25	10	Yes
Passing No. 200 sieve	0-8	10	No

APPENDIX 2

EXAMPLE SPLIT PLOT ANALYSIS FOR EVALUATION OF PHASING

APPENDIX 2. EXAMPLE SPLIT PLOT ANALYSIS FOR EVALUATION OF PHASING

This analysis is presented to provide a method which might be used to evaluate the significant factors and interactions within each stage.

The first step in the procedure is to separate the main effects and their interactions into the following three phases: (1) mixing, (2) compaction, and (3) curing. This has been done for the Indirect Tensile Strength values and is presented in Tables 13, 14, and 15.

The next step is the selection for each stage of a breakpoint below which the effects of the main effects and their interactions are considered to be negligible. The sum of squares associated with these factors and interactions is then attributable only to error.

The selection of the breakpoint is based upon the experimenter's judgment as to which factors and interactions can be considered negligible and it is usually established at a logical break in either the location and sequence of the higher factor interactions or the relative change in the magnitude of their sum of squares. In the curing stage the breakpoint was established so that all consecutive three-factor interactions located at the bottom of the listing were considered to be negligible. The sum of squares associated with the three-factor interactions were then attributed to error. The same type of approach was used in the mixing and compaction phases. The breakpoints as well as the calculation of the pooled error estimate for each are shown in Tables 13, 14, and 15.

The errors associated with the individual phases are then used to calculate the F value or variance ratio and eventually to determine the significance level of the main effects and their interactions within each stage or phase.

TABLE 13. EXAMPLE SPLIT PLOT ANALYSIS FOR MIXING PHASE

Interaction	Sum of Squares	df*	Mean Squares	F Value	Significance Level
D	90420.0	1	90420.0	819.6	1
B×D	13716.4	1	13716.4	124.3	1
A×D	13427.6	1	13427.6	121.7	1
D×F	5590.0	1	5590.0	50.7	1
F	2945.6	1	2945.6	26.7	1
C×D	2835.8	1	2835.8	25.7	1
C	2332.2	1	2332.2	21.1	1
A	2030.6	1	2030.6	18.4	5
B×C	1649.4	1	1649.4	15.0	5
A×C	785.3	1	785.3	7.1	10
A×B	618.2	1	618.2	5.6	10
A×F	592.7	1	592.7	5.4	10
B	520.0	1	520.0	4.7	10
B×F	512.8	1	512.8	4.6	10
A×D×F	201.6	1	---	---	--
C×D×F	158.8	1	---	---	--
B×D×F	40.9	1	---	---	--
C×F	40.0	1	---	---	--

*df - degrees of freedom

(continued)

TABLE 13. (CONTINUED)

Pooled Estimate of Error for Mixing a Phase

Source	Sum of Squares	df
A×D×F	201.6	1
C×D×F	158.8	1
B×D×F	40.9	1
C×F	40.0	1
	Σ = 441.3	4

$$\text{Estimate of error} = \frac{SST}{d_f} = \frac{441.3}{4} = 110.325$$

TABLE 14. EXAMPLE SPLIT PLOT ANALYSIS FOR COMPACTION PHASE

Interaction	Sum of Squares	df	Mean Squares	F value	Significance Level
G	15661.2	1	15661.2	186.2	1
E	3538.8	1	3538.8	42.1	1
D×E	2786.5	1	2786.5	33.1	1
D×G	2189.8	1	2189.8	26.0	1
E×G	1401.4	1	1401.4	16.7	1
B×F×G	1223.1	1	1223.1	14.5	1
B×D×G	829.7	1	829.7	9.9	1
B×E×G	708.6	1	708.6	8.4	5
A×E	676.9	1	676.9	8.0	5
C×E×F	644.9	1	644.9	7.7	5
A×G	570.2	1	570.2	6.8	5
E×F	355.1	1	355.1	4.2	10
C×F×G	276.8	1	---	---	--
A×D×G	226.0	1	---	---	--
A×E×G	150.0	1	---	---	--
C×E×G	103.6	1	---	---	--
C×D×G	78.6	1	---	---	--
A×E×F	71.9	1	---	---	--
A×F×G	69.5	1	---	---	--
B×E	66.0	1	---	---	--
F×G	22.7	1	---	---	--
B×G	21.6	1	---	---	--
B×E×F	4.2	1	---	---	--
C×G	2.1	1	---	---	--
C×E	.3	1	---	---	--

TABLE 14. (CONTINUED)

Pooled Estimate of Error for Compaction Phase

Source	Sum of Squares	df
C×F×G	276.8	1
A×D×G	226.0	1
A×E×G	103.6	1
C×D×G	78.6	1
A×E×F	71.9	1
A×F×G	69.5	1
B×E	66.0	1
F×G	22.7	1
B×G	21.6	1
B×E×F	4.2	1
C×G	2.1	1
C×E	.3	1
Σ =	1093.3	13

$$\text{Estimate of error} = \frac{\text{SSE}}{\text{df}} = \frac{1093.3}{13} = 84.10$$

TABLE 15. EXAMPLE SPLIT PLOT ANALYSIS OF CURING PHASE

Interaction	Sum of Squares	df	Mean Squares	F Value	Significance Level
D×H	1446.5	1	1446.5	44.8	1
A×H	1245.6	1	1245.6	38.6	1
B×F×H	645.2	1	645.2	20.0	1
E×H	429.8	1	429.8	13.3	1
C×F×H	326.4	1	326.4	10.1	5
H	303.0	1	303.0	9.4	5
C×G×H	241.8	1	241.8	7.5	5
C×D×H	217.6	1	217.6	6.7	5
G×H	206.9	1	206.9	6.4	5
C×E×H	191.6	1	191.6	5.9	5
C×H	189.6	1	189.6	5.9	5
A×E×H	173.2	1	173.2	5.4	5
B×E×H	165.2	1	165.2	5.1	5
F×H	116.5	1	116.5	3.6	10
B×H	99.8	1	99.8	3.1	10
A×G×H	89.4	1	---	---	--
B×D×H	70.5	1	---	---	--
A×D×H	17.3	1	---	---	--
B×G×H	4.3	1	---	---	--
A×F×H	1.9	1	---	---	--

(continued)

TABLE 15. (CONTINUED)

Pooled Estimate of Error for Curing Phase

Source	Sum of Squares	df
$A \times G \times H$	89.4	1
$B \times D \times H$	70.5	1
$A \times D \times H$	17.3	1
$B \times G \times H$	4.3	1
$A \times F \times H$	1.9	1
True error estimate	107.0	4
Σ =	290.4	9

$$\text{Pooled estimate of error} = \frac{290.4}{9} = 32.27$$

APPENDIX 3

SAMPLE PREPARATION AND TESTING PROCEDURE

APPENDIX 3. SAMPLE PREPARATION AND TESTING PROCEDURE

Sample Preparation:

(1) Aggregate and asphalt are heated to the appropriate mixing temperature (either 250° F or 350° F ± 5° F).

(2) Aggregate and asphalt are mixed at the specified temperature (either 250° F or 350° F ± 5° F) for 3 minutes in an automatic 12-quart-capacity Hobart food mixer at 107 rpm.

(3) All mixes are then cured at 140° F ± 2° F for 18-24 hours.

Sample Compaction:

(4) The mixes are placed in preheated ovens and brought to the required compaction temperature (either 200° F or 300° F ± 5° F).

(5) The mixes are then compacted at the specified temperature by either the Texas Gyratory-Shear or Marshall Impact Methods.

Sample Curing:

(6) The specimens are then cured for 5 days in enclosures maintained at temperatures of 40° F or 110° F ± 2° F.

Sample Testing:

(7) The specimens are removed from the curing chambers, allowed to come to a temperature of 75° F ± 2° F, and held at that temperature for 18 to 24 hours.

(8) All specimens are tested in indirect tension at a temperature of 75° F ± 2° F. An arbitrary preload (see note below) of 25 pounds is placed on the specimen prior to applying a loading rate of 2.0 inches per minute.

Note:

The preloading procedure is used for two reasons. The first is to prevent the occurrence of impact loading in the initial stages of the test. Without the preload the upper head of the loading apparatus would move downward at a rate of 2 inches per minute and would impact the specimen with a velocity of 2 inches per minute. The second reason is to minimize any seating of the loading strip with the specimen which occurs at the initial testing stage.