TEMPERATURE EFFECTS ON THE COMPRESSIONAL WAVE VELOCITIES OF ASPHALT-AGGREGATE MIXTURES

By

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CHAPTER I

INTRODUCTION

Flexible pavement design procedures are based upon theoretical, semi-theoretical, and empirical considerations modified by field observation or correlation with in-place pavement performance. The theoretical design procedures are based upon the assumption that the pavement structure behaves as a layered system of materials in which the moduli of elasticity of the respective layers decreases with depth. The lack of a rapid and dependable means of evaluating moduli values as well as other elastic constants of the materials has been a major problem in the use of theoretical design equations. In recent years vibratory techniques have shown promise of becoming rapid nondestructive methods of evaluating the elastic constants and other properties of road building materials both in the field and in the laboratory. Laboratory investigations of bituminous mixtures employing dynamic testing methods should provide a better understanding of the design requirements and better interpretation of the field performance of these materials.

In any attempt to describe the elastic behavior of a particular material an analysis of its load-deformation characteristics is needed. A plot of the amount of load required to produce a given amount of deformation, i.e., a stress-strain diagram, is a familiar method by which a particular material's elastic behavior is described.

Each material has associated with it a particular stress-strain curve. In a perfectly homogeneous, isotropic and elastic material the stress is directly proportional to the strain for all stresses. The stress-strain curve in this case is linear. However, in reality there exists no truly elastic material. The material perhaps most familiar and exhibiting the most nearly elastic behavior is steel. In steel there exists a linear relationship between stress and strain up to the "proportional limit" of the material.

The "proportional limit" is defined as the maximum stress that may be developed in the material without deviation from straight-line proportionality between stress and strain. Beyond the "proportional limit" plastic deformation occurs. Plastic deformation is defined as an increase in strain without a corresponding increase in stress. The elastic range of the curve is then that portion of the curve in which the curve is linear. The slope of the linear portion of the curve has been termed the modulus of elasticity (E) where:

$$E = \sigma/\varepsilon$$

$$\sigma = \text{stress} \qquad (1.1)$$

$$\varepsilon = \text{strain}$$

This same elastic theory has been applied extensively to other materials in the construction industry in an attempt to determine their properties and predict their behavior. In rigid paving materials such as portland cement concrete, the behavior is much less than ideally elastic. The stress-strain curve for concrete, as determined by ordinary compression tests, appears as a curvilinear line concave downward. There exists no portion of the curve that is linear. The degree of curvature depends upon the rate of loading, i.e., the slower the rate of loading, the sharper the curvature due to plastic deformation which takes place with time. In this case the modulus of elasticity as previously defined has no real meaning because its value is instantaneous due to the continuous variation in slope of the stress-strain curve. Because the material is imperfectly elastic a special definition of the E value is necessary.

The "secant" modulus of elasticity, the ratio of any given stress to the corresponding strain, is ordinarily used for concrete. However, with a rapid rate of loading the initial portions of the diagram show a more or less linear proportionality of stress to strain and the material can be considered purely elastic within this region of the curve. In this case the "initial-tangent" modulus or the slope of a line drawn tangent to the curve at the origin can be used. Within normal ranges, temperature apparently has little or no effect on the stress-strain relationships for concrete.

Most cohesionless materials are less elastic in character than concrete. They exhibit more plastic tendencies and therefore have no definite elastic limits. Similar to concrete these materials have curvilinear stress-strain relationships which are more or less time dependent and are relatively unaffected by temperature fluctuations. A "secant" modulus value termed "modulus of deformation" is arbitrarily substituted for modulus of elasticity (E) in theoretical expressions involving these materials. Due to the concave nature of the stress-strain curves the modulus of deformation decreases as unit strain increases and the magnitude of this value for a given stress or percent strain depends largely on the degree of compaction of the material.

Cohesive material such as soft clay exhibit similar behavior although the time of loading or stress application has a greater effect upon the stress-strain relationships.

Asphalt paving materials display a combinational type of behavior in that their stress-strain characteristics are related to both the asphalt and aggregate components of the mixtures. While the aggregate comprises between 90-95 % of the weight and between 80-85 % of the volume, the nature of the asphalt binder is considered to be the predominant or controlling factor in the behavior of such mixtures. In this type of material strain (deformation) increases with time. The time required for a specified deformation to occur is dependent to a large extent upon the temperature of the mixture. An increase in temperature results in a decrease in the time required for a given amount of deformation. This behavior is due primarily to the visco-elastic character is the asphalt constituent. Bituminous mixtures tend to behave brittlely at very low temperatures and they readily crack under flexure at these low temperatures. At high temperatures the mixtures exhibit plastic properties tending to viscous as the viscosity of binder is reduced. Between these temperature extremes a bituminous mixture should behave somewhat elastically or elasto-plastically at least within a certain temperature range. If this is true then for a given mixture, moduli values at various temperatures could be determined and these values should be applicable and of considerable value in the design of flexible pavement structures.

This investigation is an attempt to establish specific temperatures or at least small ranges of temperature at which the properties of asphalt-aggregate mixtures undergo a change. Information of this nature when extended and correlated with standard mechanical test results could lead to the determination of definite ranges of temperature in which an asphalt-aggregate mixture will behave plastically, elastically and as a brittle solid. Previous investigators, P. G. Manke (1)[1] and W. H. Goetz (2), have indicated that two dynamic, nondestructive test methods might be utilized for this type of investigation.

In this study a series of bituminous hot mix samples were prepared from a closely controlled, uniformly graded sand. These specimens were then tested using pulse velocity procedures by which the velocity of a compressional wave is measured in the material. According to elastic theory, there is a direct relationship between compressional wave velocity and E. Thus, some insight as to the variation of E modulus with temperature can be gained from these measurements. The mixtures were tested at temperatures ranging from -9° F to 174° F. By varying both temperature and asphalt content the effects of each upon the compressional wave velocity in the material were determined.

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[1] The numbers in parentheses correspond to the listing of the reference in the Bibliography.

CHAPTER II

REVIEW OF LITERATURE

Various nondestructive dynamic methods of testing have been used in the field of construction materials for many years. Vibrational nondestructive testing of roads and runways originated in Germany during the period from 1928 to 1939, and the publications (3,4,5) of the Deutsches Gesellschaft fur Bodenmechanik (DEGEBO) are well known to workers in this field. Initially, a mechanical oscillator of the rotary out-of-balance mass type was employed and its vibratory characteristics were studied while it was operated on many different types of soil. One practical result that emerged from these beginning investigations was a qualitative relation which was found between the velocity of propagation of the vibrations and the bearing capacity of soils reasonably uniform with depth.

Since 1948, the development of vibrational testing has been considerably influenced by the work of Van der Poel (6) and Nijboer (7) of the Dutch Shell Laboratory at Amsterdam. Van der Poel devised a machine, now known as the "Dutch Shell Vibrator," employing rotating out-of-balance masses to apply vibrational forces of up to 4 tons to the road surfaces at frequencies between 5 and 60 cycles per second. Van der Poel introduced the concept of the dynamic stiffness of the construction (4), given by $S = F_p/X$, where F_p is the force on the pavement and X is the deflection. S is a function of the mechanical properties and thicknesses of the layers forming the construction, but it is neither independent of frequency nor force. Measurements on a variety of roads however, have shown that the value of S will, under certain conditions, indicate the condition of a flexible pavement. The larger the dynamic stiffness, the stronger the construction (8).

The Road Research Laboratory in England collaborated with the Dutch Shell Laboratory in making vibrational experiments on a number of British roads (9) and later developed an apparatus to measure the phase velocity of vibrations over a much wider frequency range (up to 30 kilocycles per second). This development enabled data to be obtained for estimating the quality of the various layers of material incorporated in a road. The objectives of this work were to develop tests for assessing the quality of existing roads and to obtain information concerning elasticities of various road materials for later use in computing stresses under moving vehicles.

Determination of the modulus of elasticity using sonic or resonant frequencies has for a number of years been applied extensively in the concrete industry. This method of nondestructive testing of concrete has been used for surveys of cracks in massive hydraulic structures, durability or condition surveys of old structures, the study of early strength development in fresh concrete, and laboratory studies involving its elastic and mechanical properties (10).

Within the last few years this method of testing has been applied to the study of bituminous mixtures. While a considerable number of experimental and theoretical studies of the propagation of vibrations in pavement structures have been made, the development of dynamic testing techniques has been slow. This is due primarily to the difficulties of interpreting the relationships between velocity of propagation and frequency of vibrations generated at the surface with the dynamic modulus of elasticity and thicknesses of the layer. Divergence of opinion among the respective investigators on the interpretation of results in terms of the mechanical properties of materials has also been a contributing factor to the slow development of these techniques (8).

In the sonic method of testing, the fundamental or resonant frequency of the specimen is determined and Young's modulus (E) is calculated from Equation 2.1:

$$E = CWn^2 \tag{2.1}$$

where:

E = Young's modulus

W = weight of specimen

n = resonant frequency of specimen

C = a factor which depends on the shape and size of the specimen, the mode of vibration and Poisson's ratio.

The problem in this method of testing is determining the value of C. Hong (11) determined that this factor is dependent upon the thickness to length ratio (t/1) of the specimen and Poisson's ratio of the material.

One of the earliest known applications of sonic test procedures to bituminous mixtures was made by Davidson and Strauss (12) in 1949, under the direction of Dr. L. F. Rader at the University of Wisconsin. Davidson and Strauss concluded that the modulus of elasticity of asphaltic paving mixtures chilled to low temperatures could be determined using this method of testing. They also concluded that at room temperature the sonic test method was not applicable for determining E. They further showed that as the density and asphalt content of the specimen was increased the sonic modulus of elasticity increased.

W. H. Goetz (2), in 1955, stated that even though the modulus of elasticity values calculated from the fundamental resonant frequency measurements with the aid of elastic theory may not be strictly valid, particularly at temperatures above 40° F, such measurements did provide valuable information concerning the elastic-plastic characteristics of bituminous-aggregate mixtures. Goetz's measurements seemed to be sufficiently valid at lower temperatures so as to provide a measure of stiffness of the mixtures. Goetz concluded: that as the temperature decreases, sonic modulus of elasticity increases; that there is no consistent relationship between the sonic test values and asphalt content; and that the influences of the penetration grade of the asphalt cement is very small.

Other work by Goetz and O. B. Andersland (13) determined that the dynamic sonic test was a quick, efficient method of evaluating the stripping qualities of compacted bituminous mixtures. Specimens used in this study contained materials of the same kind, gradation, and proportions and were compacted in a manner similar to that used in actual construction. The results of this work indicated that the sonic test had some definite advantages over both the Immersion-Compression Test (ASTM No. D 1075 - 54) and the Visual Stripping Test (ASTM No. D 1664 - 66T) methods of evaluation.

Bawa (14), in a master's thesis written at Purdue University, determined that a bituminous mixture could be considered as an elastic-plastic material capable of showing both elastic as well as plastic and viscous behavior. Bawa stated that temperature had a decided effect on both Poisson's ratio and the modulus of elasticity values of bituminous mixtures. He proposed that changes in the modulus of elasticity with temperature indicate the existence of a transition zone in which the mixture changes from a plastic to an elastic state as the temperature decreases.

Pulse velocity techniques of determining Young's modulus are thought to have several inherent advantages over the resonant frequency method. First, pulse velocity measurements are independent of size or shape of the specimen. Secondly, pulse velocity measurements are easily and rapidly performed. In addition, Long, et al (15), reported good agreement of dynamic modulus values computed from pulse velocities in concrete with those determined from static flexural tests.

P. G. Manke and B. M. Gallaway (1) used the pulse velocity measurement technique on various types of flexible pavement construction materials. Their findings relative to bituminous mixtures indicated: the visco-elastic character of the binder influences the wave propagation; wave velocities increase with increased asphalt content up to an expected limit; and that wave velocity decreases with increasing temperature. They concluded that using pulse velocity techniques it might be possible to define the temperature ranges in which the brittle, elastic, plastic, and viscous properties of a bituminous mixture are dominant. This investigation is an attempt to delineate these temperature ranges for a series of asphalt-aggregate mixtures using similar measurement techniques.

CHAPTER III

THEORY OF ELASTIC WAVE PROPAGATION

The theory of transmission of impulses through a solid body has been studied extensively in connection with the propagation of earthquake waves through the earth. In an extended solid, any generated impulse can be shown to separate into two groups of waves: a longitudinal or compressional wave, in which the particles vibrate in a direction parallel with the direction of wave transmission; and a transverse or shear wave, in which the particles vibrate in a direction perpendicular to the direction of wave transmission. In some instances a surface wave may be generated at the interface between two different media, either on a free surface or in a layered system. These surface waves can also be divided into classes: Rayleigh waves, in which the general mode of vibration is an elliptical motion in the vertical plane in the direction of wave transmission; and Love waves, in which the mode of vibration is transverse to the direction of propagation in the horizontal plane (16). Compressional and shear waves travel with a velocity determined by the elastic constants and the density of the medium. The longitudinal or compressional wave travels with the greatest velocity.

The motion that is produced in an elastic body by a suddenly applied force is not transmitted immediately to all parts of the body. Initially, the most remote parts of the body are undisturbed and the deformations that are produced by the applied stress are transmitted through the body in the form of elastic waves. The theory of the propagation of these waves is developed from the fundamental static stress-strain relationships of elastic bodies. The theoretical development of the equations of motion of particle displacement in the propagation of the two types of waves through a homogeneous, isotropic, elastic medium can be found in most texts on elastic theory (17), seismic waves (18), or sound transmission (19). These equations, in terms of partial displacement and the three coordinate directions, are:

$$(\lambda+2\mu)\frac{\delta^2 U_x}{\delta x^2} + \mu\left(\frac{\delta^2 U_x}{\delta y^2} + \frac{\delta^2 U_x}{\delta z^2}\right) + (\lambda+\mu)\left(\frac{\delta^2 U_y}{\delta x \delta y} + \frac{\delta^2 U_z}{\delta x \delta z}\right) = \rho\frac{\delta^2 U_x}{\delta t^2}$$

$$(\lambda+2\mu)\frac{\delta^2 U_y}{\delta y^2} + \mu\left(\frac{\delta^2 U_y}{\delta x^2} + \frac{\delta^2 U_y}{\delta z^2}\right) + (\lambda+\mu)\left(\frac{\delta^2 U_x}{\delta x \delta y} + \frac{\delta^2 U_z}{\delta y \delta z}\right) = \rho\frac{\delta^2 U_y}{\delta t^2} \quad (3.1)$$

$$(\lambda+2\mu)\frac{\delta^2 U_z}{\delta z^2} + \mu\left(\frac{\delta^2 U_z}{\delta x^2} + \frac{\delta^2 U_z}{\delta y^2}\right) + (\lambda+\mu)\left(\frac{\delta^2 U_x}{\delta x \delta y} + \frac{\delta^2 U_y}{\delta y \delta z}\right) = \rho\frac{\delta^2 U_z}{\delta t^2}$$

where U_x, U_y, and U_z are the components of displacement of a point from its rest position in the three coordinate directions. The Lamé coefficients, λ and μ greatly simplify the notation and are related to the more familiar elastic constants by the following relationships:

$$\lambda = \frac{\nu E}{(1+\nu)(1-2\nu)} \quad \text{and} \quad \mu = G = \frac{E}{2(1+\nu)} \qquad (3.2)$$

where:

ν = Poisson's ratio

E = Young's modulus

G = shear modulus

ρ = density of the medium.

If the waves are considered to be plane waves, the two different types of transmission can be illustrated quite simply.

First, assume that the wave is propagated in the X direction and that all displacement is parallel to the X axis, i.e., U_y and U_z are both zero, and that U_x is a function of x and t only. Then equations 3.1 reduce to

$$(\lambda+2\mu)\frac{\delta^2 U_x}{\delta x^2} = \rho\frac{\delta^2 U_x}{\delta t^2} \tag{3.3}$$

This is the equation for the longitudinal or compressional wave in which all particles are moving parallel to the direction of wave propagation.

Second, assume that the wave is propagated in the X direction, all displacement is parallel to the Y axis, i.e., U_x and U_z are zero, and that U_y is a function of x and t only. Then equations 3.1 reduce to

$$\mu\frac{\delta^2 U_y}{\delta x^2} = \rho\frac{\delta^2 U_y}{\delta t^2} \tag{3.4}$$

This is the equation for the transverse or shear wave in which the particle motion is perpendicular to the direction of travel.

For both kinds of waves the equations of motion have the common one-dimensional wave equation form

$$\frac{\delta^2 U}{\delta t^2} = C^2 \frac{\delta^2 U}{\delta x^2} \tag{3.5}$$

in which C is the velocity of propagation of the wave and has the form

$$C = V_c = \sqrt{\frac{\lambda+2\mu}{\rho}} = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \tag{3.6}$$

for the case of the compressional waves, and

$$C = V_s = \sqrt{\frac{\mu}{\rho}} = \sqrt{\frac{E}{2\rho(1+\nu)}} \tag{3.7}$$

for the case of the shear waves.

The relation between compressional, shear, and Rayleigh wave velocities and Poisson's ratio in a semi-infinite elastic medium is shown in Figure 1.

Equations 3.6 and 3.7 show the relationship of the shear and compressional wave velocities in an isotropic elastic solid to the density and elastic constants of the media. These equations have been employed directly or in a slightly modified form by many investigators to compute the dynamic modulus of elasticity of various materials. The use of these equations is based on the assumption that the materials behave as perfectly elastic solids. This is not strictly true as most materials will exhibit plastic as well as elastic properties.

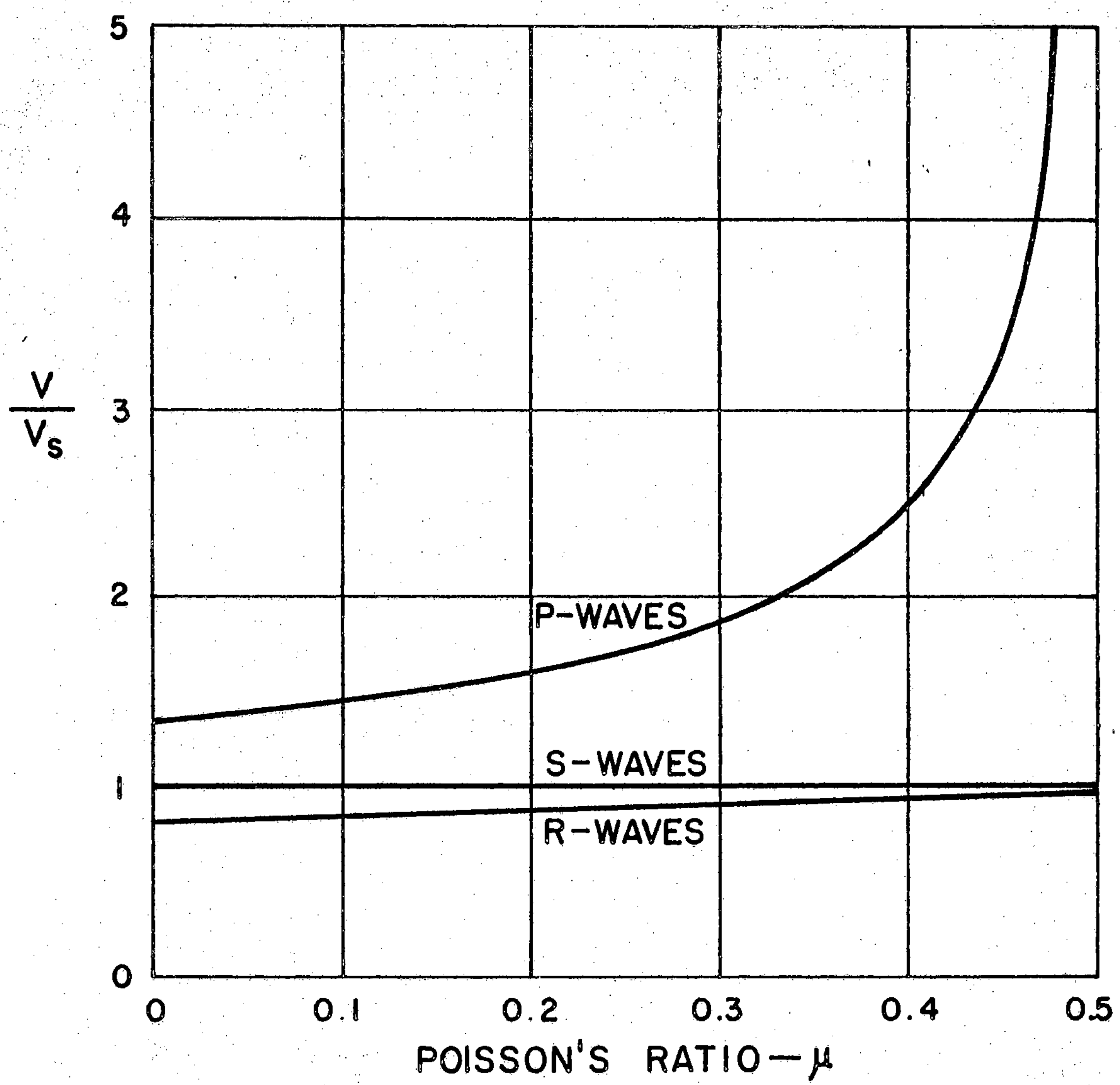


Figure 1. Relation Between Poisson's Ratio (μ) and Velocities of Propagation of Compression (P), Shear (S) and Rayleigh (R) Waves in a Semi-Infinite Elastic Medium

Using the above equations, the "dynamic" modulus of elasticity of highway materials can be calculated if the compressional wave velocity, Poisson's ratio, and the density of the medium can be determined. Compressional wave velocity can be determined by the processes described in this paper and becomes readily obtainable.

The density of a material can be readily determined by standard methods and presents no problem in the calculations. In fact, the density could be estimated in most cases since a small variation in its value has little effect on E (20).

Poisson's ratio varies over a small range for most elastic materials. As in the case of density, small differences do not greatly affect the results (20) and in many cases it may be estimated with sufficient accuracy to be used in the analysis. However, for greater accuracy and because of the wide range of strengths or stabilities of the materials being tested, Poisson's ratio should be measured directly.

Several methods for evaluating Poisson's ratio in soils and granular materials are as follows:

1) Coefficient of earth's pressure at rest test (21).
2) Variable confining pressure test (22).
3) Resonant frequency test (23).
4) Pulse velocity measurements

The first two methods utilize the triaxial test to develop mathematical or graphical relationships that can be used to find Poisson't ratio. The coefficient of earth pressure at rest method is a rather complicated procedure that involves the determinations of pore pressures within the material specimens. The variable confining pressure test uses a radial stress that always varies linearly with the vertical stress. Synchronization of the two stresses is a major problem in this method. Because of the nature of these tests and difficulties in making the measurements they appear to be less favorable than the latter two methods in regard to the testing of highway materials.

Method 3 involves the determinations of the dynamic modulus of rigidity (G) and the dynamic Young's modulus (E) from the fundamental torsional and longitudinal frequencies of the test specimen. Poisson's ratio is then calculated from the following relationship:

$$\nu = \frac{E}{2G} - 1 \tag{3.8}$$

While this standard test is dynamic in nature, the procedures and equipment differ considerably from those of Method 4.

In relation to the pulse velocity technique of measurement, it appears that the compressional and shear wave velocities of a material specimen can be utilized for determining Poisson's ratio as well as E values.

The compressional wave velocity determination presents no major problem in this procedure. By the use of an axial expander type piezoelectric crystal to induce a train of longitudinal waves along an axis of the specimen, the travel time of the wave train can be determined using electronic equipment. Since the distance of travel is easily measured then the wave velocity can be calculated.

In determining the shear wave velocity, a radial expander type of piezoelectric crystal is used to induce a train of dilational waves that propagate along an axis of the specimen. The travel velocity of this type of wave should be determinable in a manner similar to that of the compressional wave. Specific information as to the effectiveness of this procedure is lacking. However, McSkimim (24) has been able to determine the amplitude and phase variations of shear waves in cylindrical rods using Y - cut quartz crystals as the transmitting and receiving piezoelectric transducers. It seems reasonable to assume that the velocity of travel could also be determined.

If such a procedure can be utilized to determine the compressional wave velocity (V_c) and the shear wave velocity (V_s), Poisson's ratio of a material specimen can be calculated from the following relationship by combining Equations 3.6 and 3.7:

$$\frac{V_c}{V_s} = \sqrt{\frac{1-\nu}{\frac{1}{2}-\nu}} \qquad (3.9)$$

after rearranging:

$$\nu = \frac{1-\frac{1}{2}\left(\frac{V_c}{V_s}\right)^2}{1-\left(\frac{V_c}{V_s}\right)^2} = \frac{1}{1-\left(\frac{V_c}{V_s}\right)^2} + 1 \qquad (3.10)$$

In subsequent work extending this study, Method 4 will be investigated more fully.

CHAPTER IV

TESTING DETAILS

Equipment

Electronic Equipment

The electronic equipment used to determine compressional wave velocities consisted of a pulse generator, source and receiver transducers and an oscilloscope. This equipment is shown in Figure 2.

The pulse generator operated on an 110 volt, 60 cycle, AC current. It provided an 110 volt, 60 cycle per second DC spike pulse to the source transducer by the discharge of a condenser through a thyratron tube. The pulse generator also actuated a horizontal electron beam trace on the face of the cathode ray tube by providing an 11 volt DC signal to the triggering circuit of the oscilloscope. This occurred at the same instant that the compressional pulse was transmitted to the specimen. A schematic wiring diagram of the pulse generator is shown in Figure 3.

The piezoelectric transducers were 1.0 inch diameter by 0.27 inch thick lead titanate zirconate ceramic discs with a resonant frequency of approximately 260 kilocycles per second. These transducers are available commercially through the Clevite Corporation. Both the source and receiver transducer assemblies were constructed in the same way. A ceramic disc was cemented between two brass discs of the same size with an epoxy cement and then a solid lucite cylinder of the same

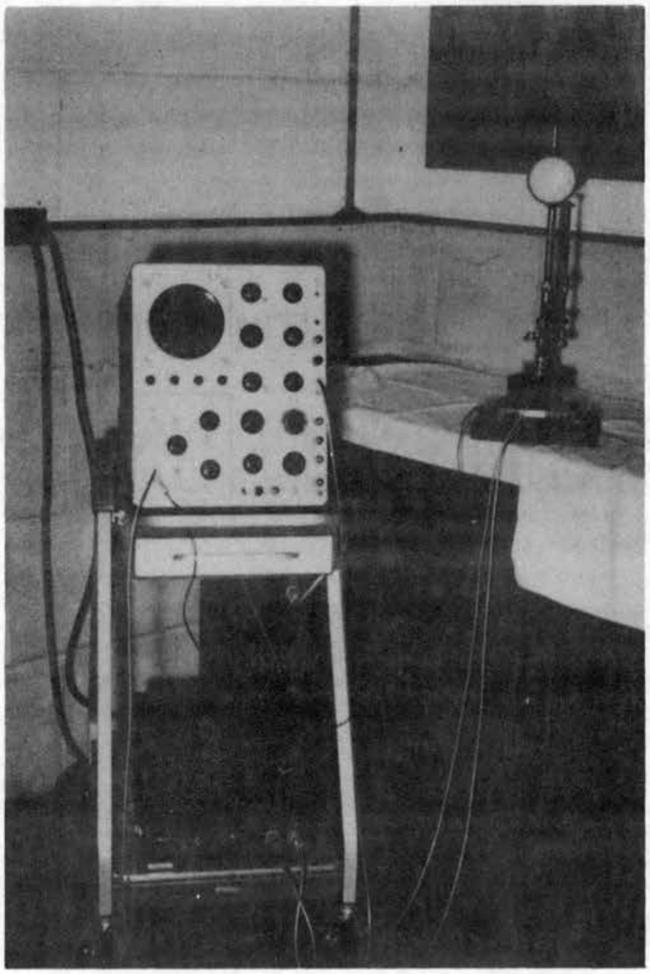


Figure 2. Testing Apparatus

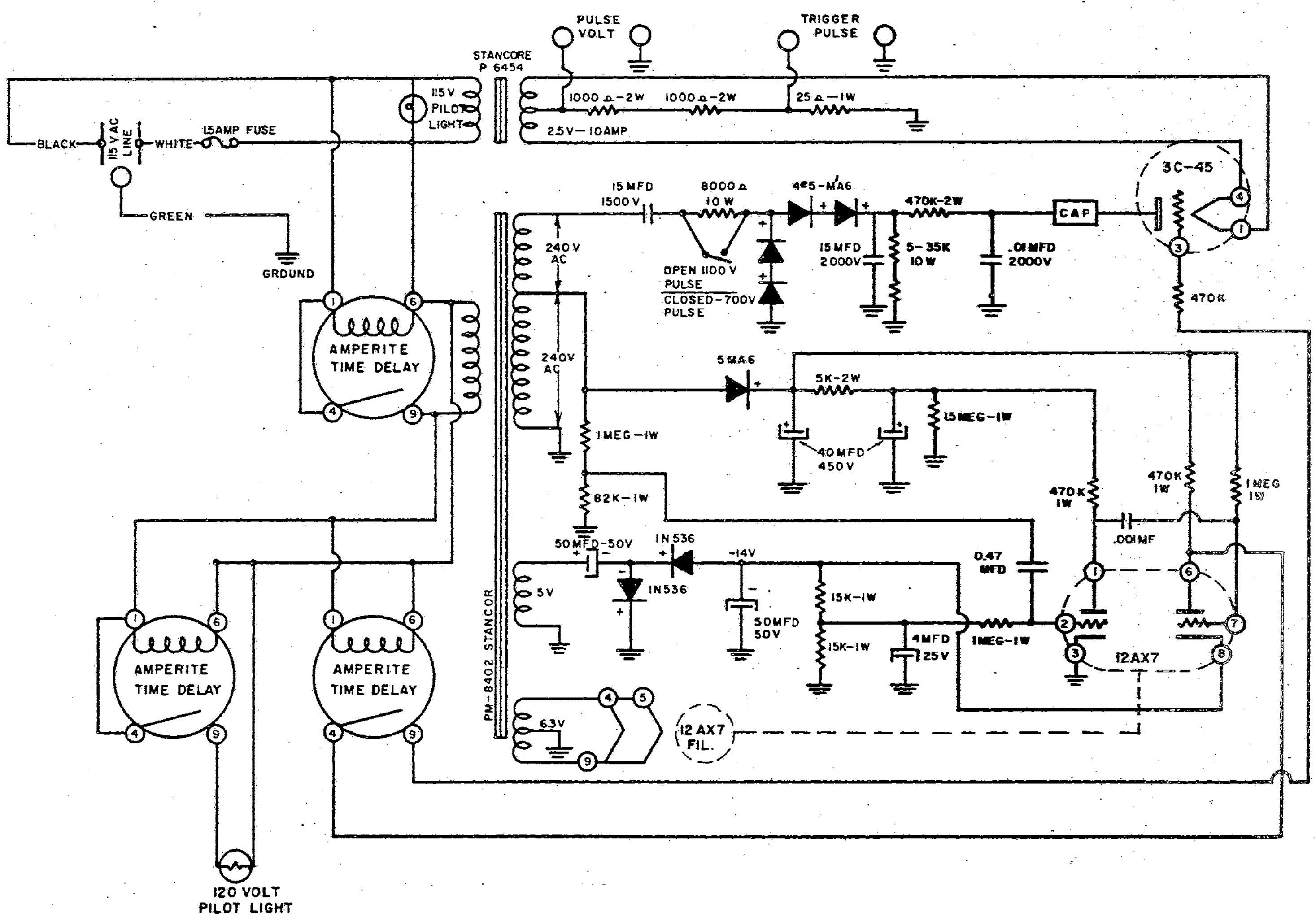


Figure 3. Schematic Wiring Diagram of Pulse Generator

diameter was cemented to one face of the assembly. The entire transducer assembly was 1.0 inch in diameter and 1.75 inches tall (Figure 4).

The brass discs served as terminal electrodes of the assembly with the brass disc between the lucite and the ceramic disc as the positive terminal and the other brass disc as the negative or ground terminal. The face of the brass ground terminal of the source transducer was placed against the bottom face of the test specimen to protect the piezoelectric ceramic and to prevent current flow through the test material. The receiver transducer was also placed with the ground terminal in contact with the test material, directly opposite the source transducer on the top face of the specimen.

An electrical pulse was then applied across the source transducer exciting thickness modes of vibration in the ceramic. The vibrating transducer thereby imparted a compressional wave into the bottom of the test specimen. When this wave reached the top of the specimen the particle disturbance excited the piezoelectric transducer which returned the transmitted mechanical energy to electrical energy. The receiver transducer was connected directly to the vertical input connection on the oscilloscope so that when the electrical signal arrived it was amplified and displayed on the oscilloscope screen causing a vertical deflection of the trace.

The transducers were positioned and held in place by the use of the apparatus shown in Figure 5. This apparatus insured that the transducers were placed directly opposite each other at the approximate center of the specimen. The apparatus also allowed rapid measurement of specimen height and generally speeded up the testing operation. Figure 6 shows the apparatus with a specimen in place for testing.

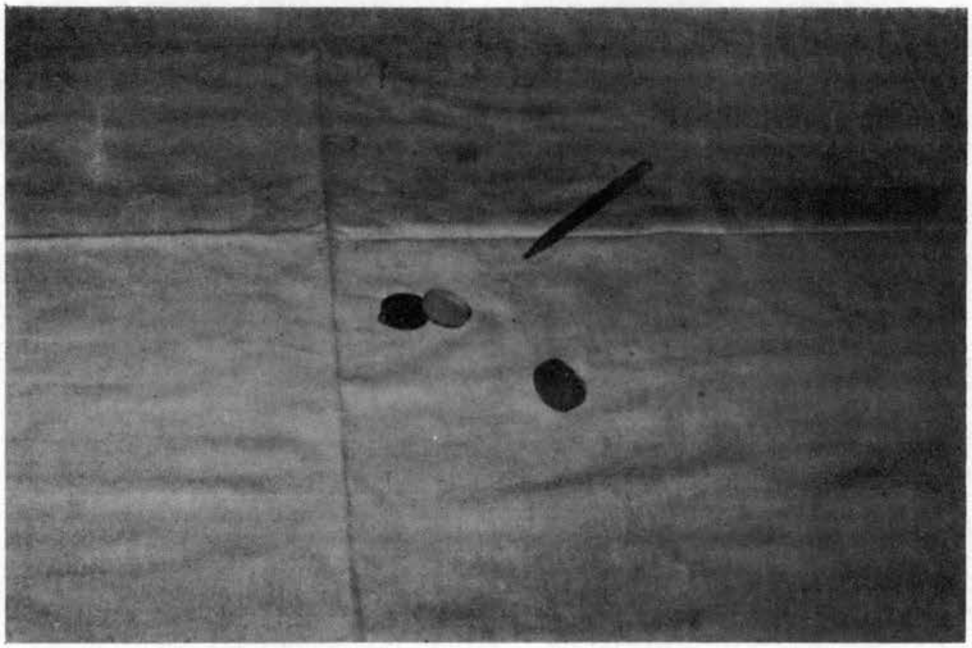


Figure 4. Ceramic Transducer and Brass Terminals

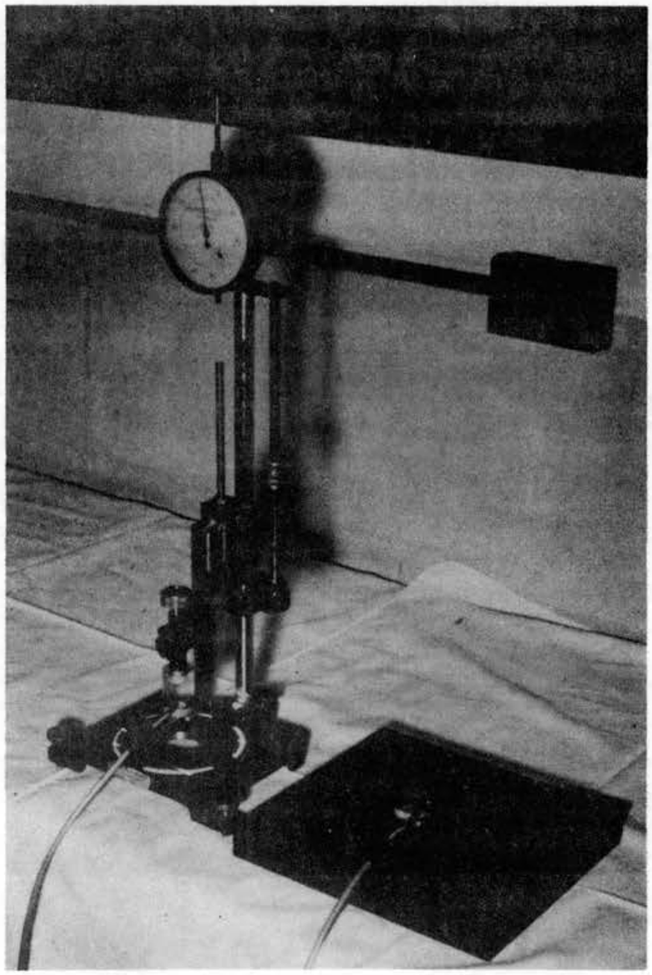


Figure 5. Positioning Apparatus with Transducer Assemblies

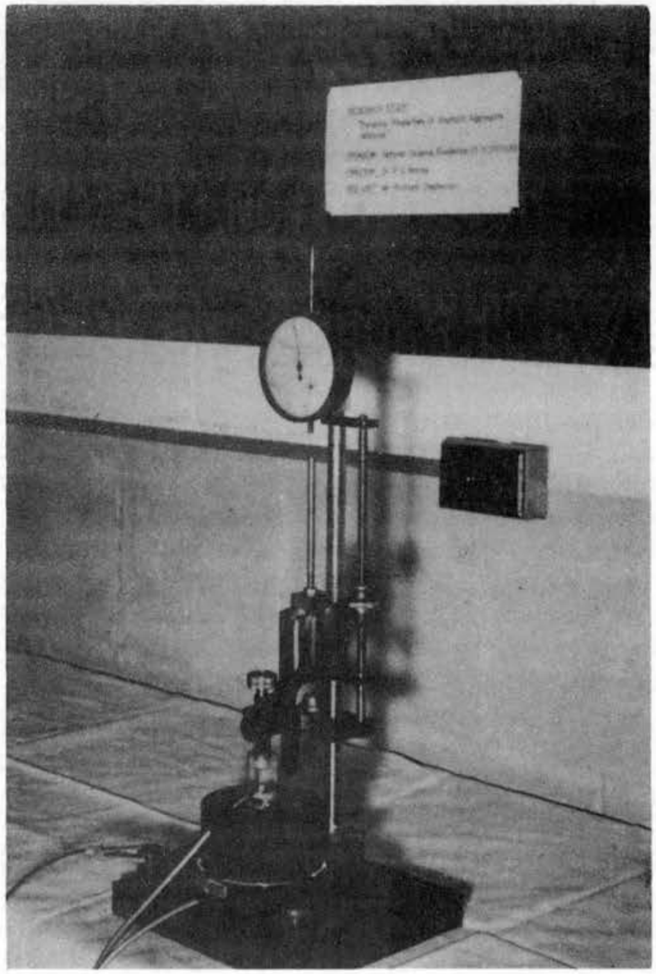


Figure 6. Positioning Apparatus with
Specimen In Place

The oscilloscope was a Tektronix type 545B unit with a type B wide-band, high gain, preamplifier unit. This instrument features two time base generators by which delayed sweep operations for highly accurate time measurements can be accomplished. With the preamplifier unit, the oscilloscope has a calibrated vertical deflection sensitivity of 0.005 volts per centimeter to 20 volts per centimeter and a horizontal time sweep range of 2 microseconds per centimeter to 1 second per centimeter with an accuracy within ± 3%. The rise time of the amplifier is 18 nanoseconds (1 x 10^{-12} seconds).

Environment Controlling Equipment

A Lab-Line Model 3922 controlled temperature cabinet with an operating range of +150° F to -150° F was used to reduce the temperature from room temperature to -9° F in the specimens. This cabinet is shown in Figure 7.

The heating of the specimens above room temperature was done in a Blue M Model OV-520C-1 forced draft oven with an operating range of 38° C to 260° C. Figure 8 shows this oven.

Sample Preparation Equipment

The aggregate particles were sized using U.S. standard sieves and a Ro-Tap Testing Sieve Shaker. The mixing of the asphalt with the aggregate material was accomplished using a Hobart Mixer Model C-100 equipped with a whip beater.

A motorized gyratory-shear molding press (Figure 9) was used to compact the specimens. The compactive effort is applied by hydraulic pressure and rotation of the compaction mold held in an inclined position until one stroke of the pump handle caused a pressure indica-

Figure 7. Controlled Temperature Cabinet

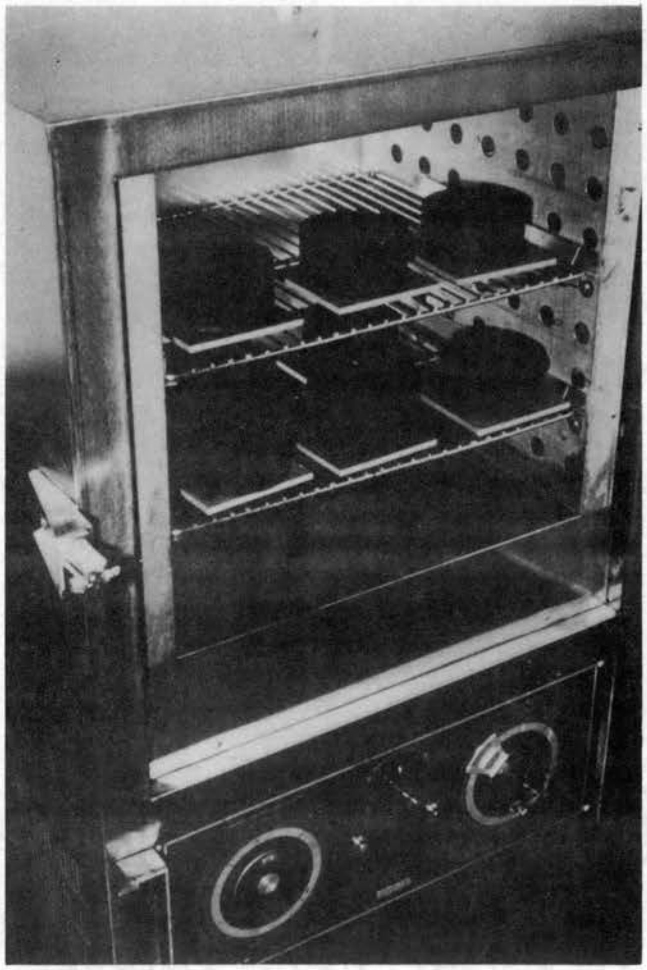


Figure 8. Forced Draft Oven

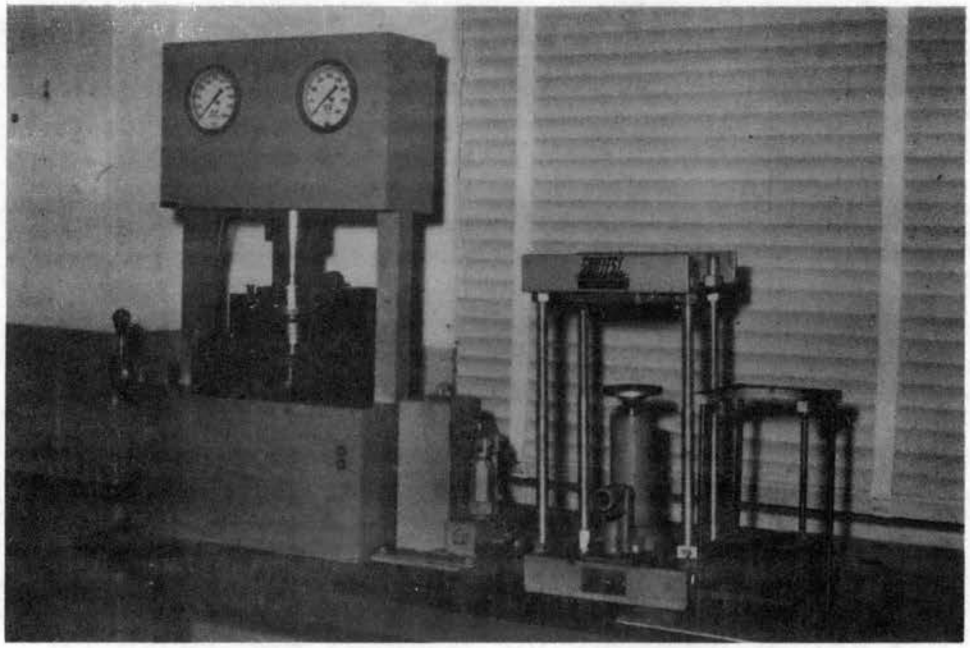


Figure 9. Motorized Gyratory-Shear Compactor

tion of 150 psi or more. When this end point was reached, an additional total pressure of 2500 psi was applied with the mold in a horizontal position and the pressure was then slowly released.

Hveem Test Equipment

A standard Hveem stabilometer and a 30,000 pound Versa Tester hydraulic testing machine, Model AP-1000, were used to determine the relative strength or stability of the specimen after the compressional wave velocity tests were completed.

Materials

Sand

A well-graded river sand was utilized in the asphalt-aggregate mixtures studied in this investigation. This material meets the Oklahoma Highway Department specifications for sand-asphalt mixtures and was obtained from a source on the North Canadian River in Oklahoma City, Oklahoma. The sand is shown in Figure 10.

Both wet and dry sieve analyses were run on this material and the results are given in tabular form below.

TABLE I

SIEVE ANALYSIS

Sieve Size	% Passing	
	Wet	Dry
3/8	100.0	100.0
#4	87.6	86.0
#10	83.4	76.4
#40	65.3	39.6
#80	30.1	10.1
#200	16.7	3.0

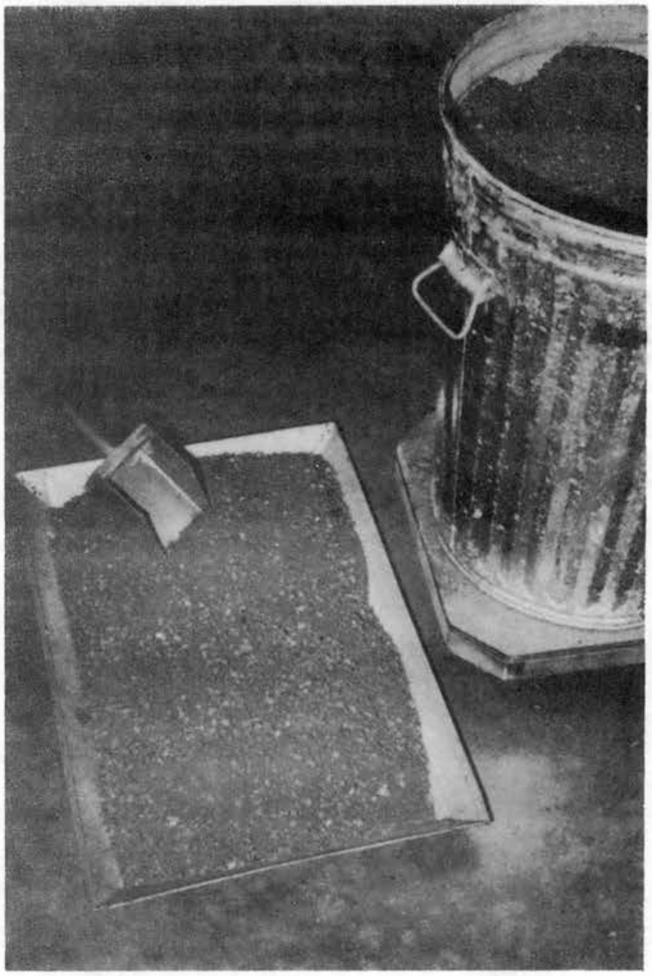


Figure 10. River Sand Used in the Mixtures

A standard hydrometer analysis was also run on the material passing the #200 sieve. These results, along with the standard sieve analysis data are presented in Figure 11.

Asphalt

Two different asphalts were used in this test. The first is a 60 - 70 penetration grade, steam and vacuum refined material. The second is an 85 - 100 penetration, solvent refined material. The properties of the two different asphalt coments are given in Table II.

TABLE II

PROPERTIES OF THE ASPHALT CEMENT

Property	Asphalt 1	Asphalt 2
Penetration (77° F)	70	96
Specific Gravity (77° F)	0.999	1.002
Softening Point	118°F	117°F
Viscosity[2] Megapoises		
50° F	*	30.0
77° F	*	0.72
Viscosity[3] Poises		
140° F	*	1605.0
275° F	*	3.2

*Viscosity information on Asphalt 1 was not available.

[2]Measurements made using sliding plate, microfilm viscometer and viscosities calculated at 5 x 10^{-2} rate of shear.

[3]Measurements made in Cannon-Manning Vacuum capillary tube viscometer.

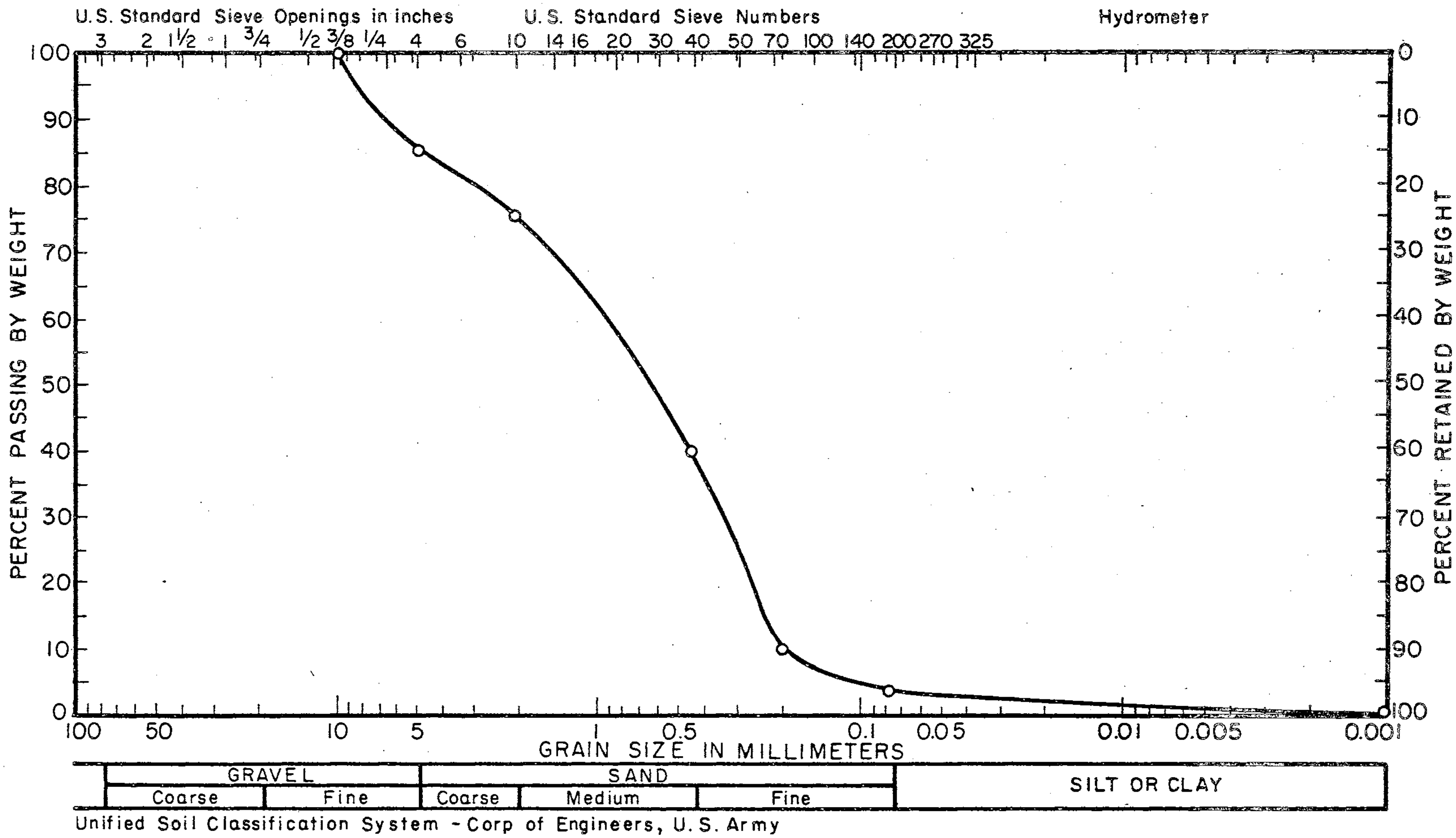


Figure 11. Grain Size Distribution of Natural Material

Sample Preparation

The sand was oven dried to a constant weight. To insure uniformity, all particles greater than 3/8 inch were removed and strict control of the particle sizes incorporated in the mixture was maintained. The sandy material was broken into sieve fractions as indicated in Table III. The respective fractions were then recombined in the proportions shown to produce the aggregate gradation used in the mixtures (see Figure 12).

TABLE III

PARTICLE SIZE COMPOSITION

Sieve Size		Percentage of Total Sample
3/8" - #4		14.0
#4 - #10		9.6
#10 - #40		36.8
#40 - #80		29.5
Minus #80		10.1
	Total	100.0 percent

For each different asphalt cement used, fifteen test specimens were prepared incorporating five different asphalt contents. These asphalt contents used were 4, 5, 6, 7, and 8 percent by weight of mixture. For a particular asphalt content, enough aggregate was recombined (approximately 2700 grams) for three specimens of approximately 900 grams each.

Both the sand and asphalt cement were heated prior to mixing.

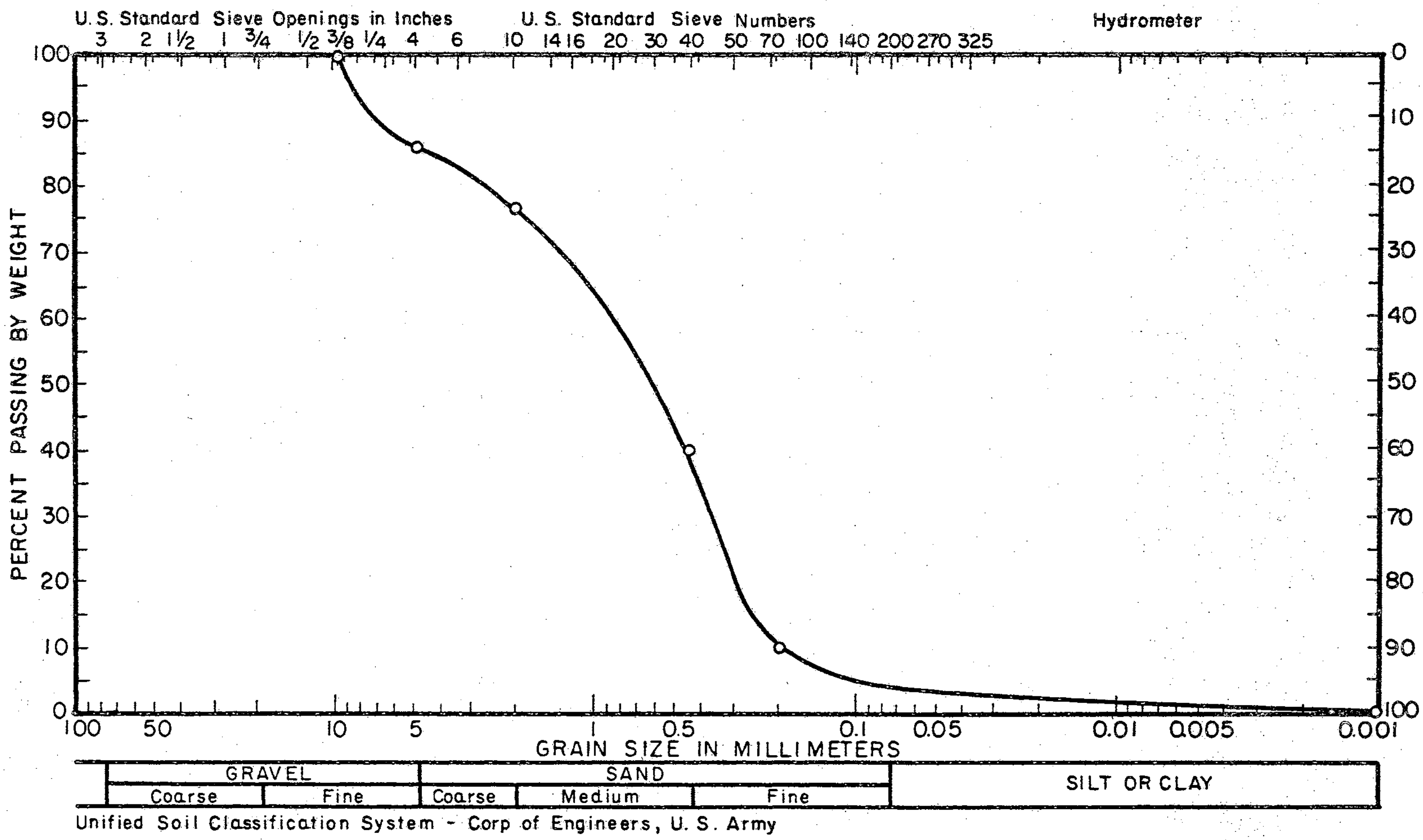


Figure 12. Grain Size Distribution of Recombined Material

The sand was heated to 275° F and the asphalt cement to 250° F. These temperatures gave the asphalt cement a good mixing viscosity and permitted uniform coating of the aggregate particles. The heated aggregate was placed in a mixing bowl. The required amount of asphalt cement was then weighed into the bowl and the mixing accomplished using the Hobart mixer.

The mixture was then compacted into specimens four inches in diameter and approximately two inches high using the gyratory-shear compactor. These specimens were made using the Texas Highway Department method of compacting test specimens of bituminous mixtures (Test Method Tex-206-F, Part II, Tentative, June, 1964). This method imparts a kneading action to the mixture being molded and compactive effort is applied until the densified mixture has a certain resistance to load. The compacted specimens are shown in Figure 13.

Testing Procedure

The actual determination of compressional wave velocity was a simple procedure. A measurement of the time required for the wave to travel through the specimen was determined and then, knowing the specimen length, the velocity was calculated as the ratio of length to travel time. The specimen was placed in the testing apparatus with its bottom surface resting on the source transducer and the receiver transducer was positioned in the center of its top surface. To insure good contact with the specimen the transducers were lightly coated with silicone grease. The thickness of the specimen was then measured using the gage on the assembly. The oscilloscope was triggered at the same time voltage was applied across the source transducer

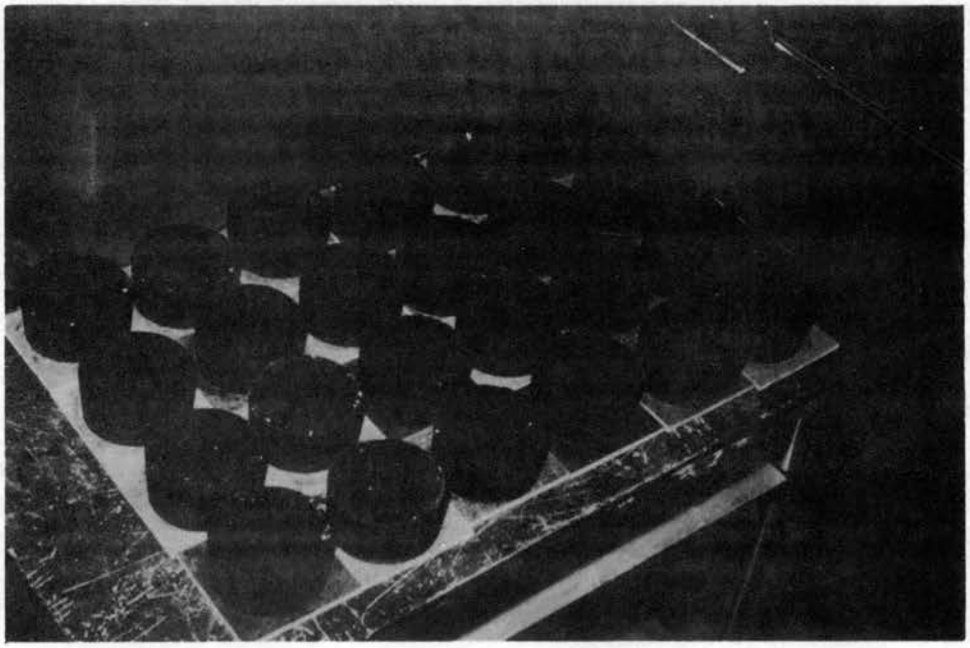


Figure 13. Compacted Test Specimen

by the pulse generator. When the oscilloscope was triggered it caused an electron beam to travel horizontally across the oscilloscope screen from left to right. When the compressional wave reached the receiver transducer it generated a voltage causing the beam to deflect upward. The travel time of the compressional wave through the specimen was then measured as the horizontal portion of the trace from beginning of the trace to the beginning of the first vertical deflection or "first arrival." Figure 14 illustrates the respective equipment components and the testing arrangement schematically.

The trace appeared on the oscilloscope screen as a steady display. The travel time was determined by moving the display to the left and reading the time per centimeter of length of the initial horizontal portion of the trace directly from the oscilloscope dials. This time was the gross travel time, i.e., the time required for the wave to travel through the transducer assemblies, as well as the specimen itself. Therefore the actual travel time was determined by subtracting a "transducer constant" from the gross travel time. This "transducer constant" was the time required for the wave to travel through the transducer assemblies.

This constant was determined by placing the transducers face to face and measuring the travel time as described above. For the set of transducers used throughout this study the transducer constant was 2.85 microseconds. As a check on this value the travel times corresponding to first arrivals in steel specimens of different lengths were determined (Figure 15). By plotting these values against length of specimen and extrapolating to zero length the transducer constant was determined. This value was in good agreement with the direct measured

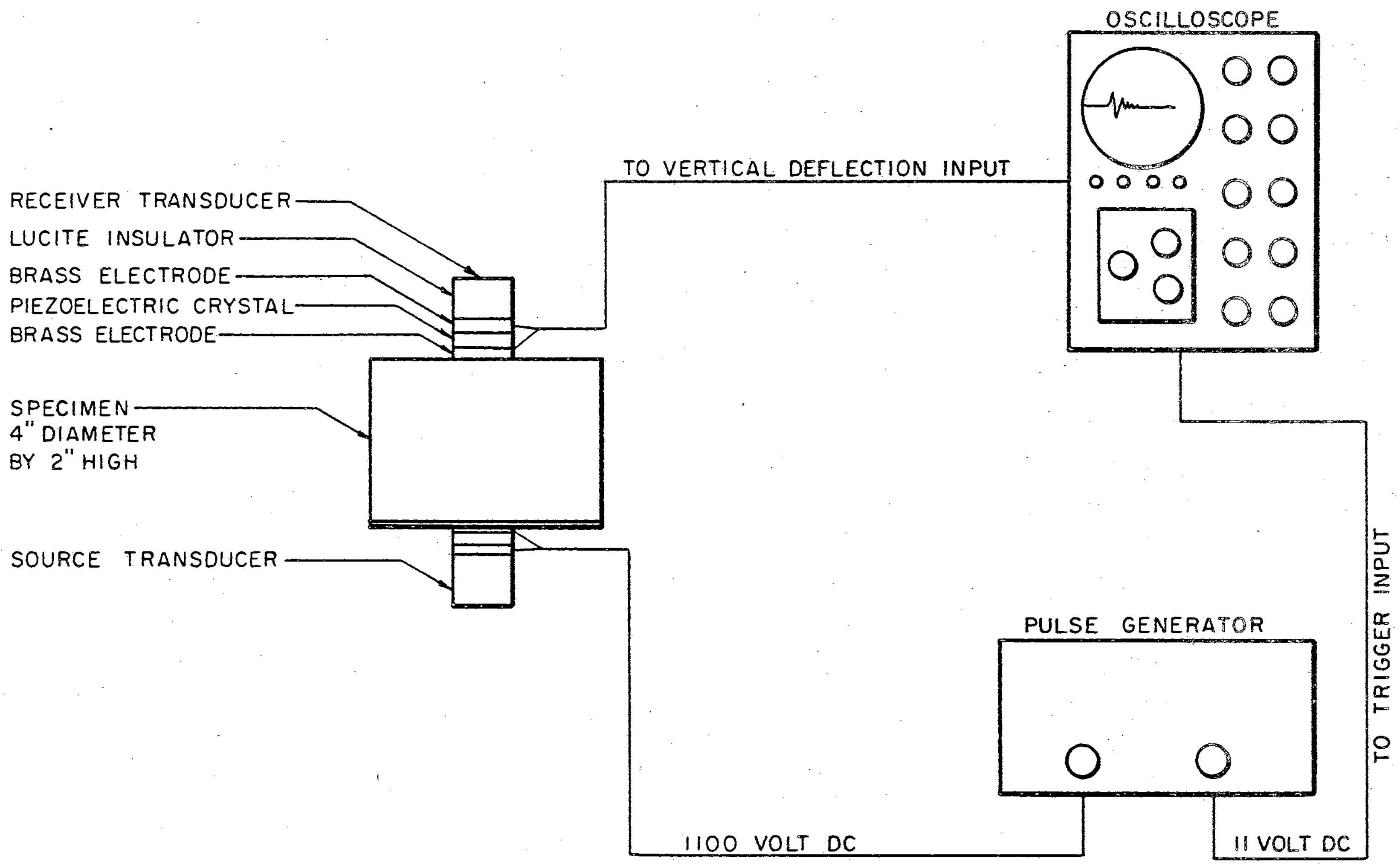


Figure 14. Schematic Diagram of Equipment Components and Test Arrangement

Figure 15. Steel Test Specimens

value. Table IV gives the length and travel times in the steel specimens and Figure 16 shows the plot of these measurements and their extrapolation to zero length.

The asphalt-aggregate specimens were tested for their compressional wave velocities at temperatures varying from -9° F to 170° F. The specimens were first tested at 70° F (room temperature) and then the temperature of the specimen was reduced in increments of approximately 5° F with travel time readings taken at each increment until a temperature of approximately -10° F was reached. The specimens were then allowed to return to room temperature and the testing procedure was repeated as the temperature was increased from room temperature to 170° F.

TABLE IV

STEEL STANDARD OBSERVATIONS

Length of Steel, in.	Gross Travel Time, microseconds	Velocity (fps)
2.0	11.12	15,000
4.0	20.22	16,450
6.0	28.45	17,600
8.0	37.45	17,800

Because of the short time required for these tests, it was felt that the temperature of the specimens would change very little during the period when they were removed from the temperature cabinet and tested. However, the opening of the cabinet door caused a change in the temperature in the cabinet which resulted in inaccurate temperature readings. To overcome this program an unused specimen was cored to

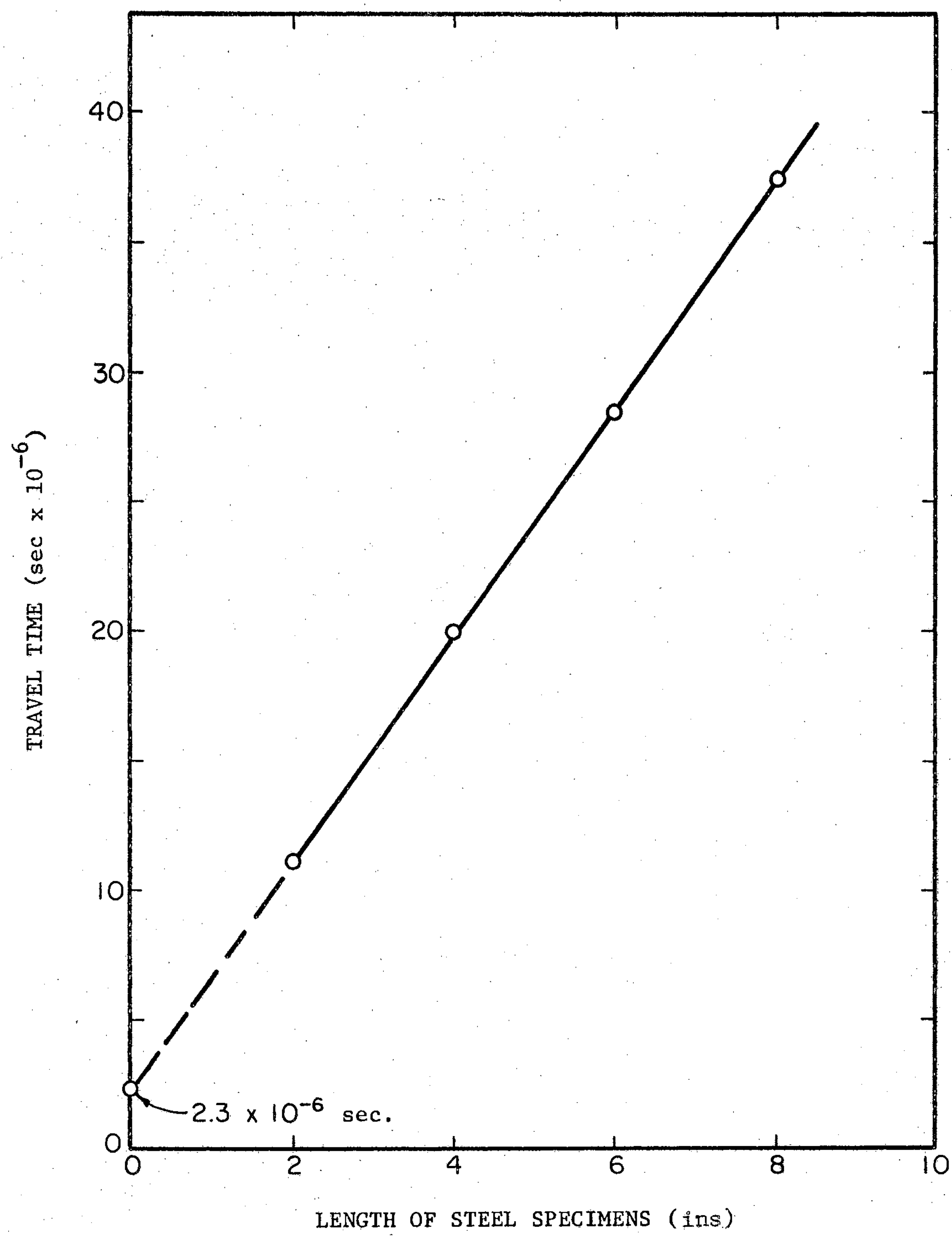


Figure 16. Determination of Transducer Constant by Extrapolation of Steel Specimen Observations

mid-depth and a thermometer accurate to 0.1° C was inserted. After this procedure, the hole was sealed with asphalt (see Figure 17). This specimen was then subjected to identical environmental conditions as were the test specimens. The temperatures given by this arrangement were considered indicative of the test specimen temperatures.

A photograph of two typical oscilloscope traces that were obtained with a Tetronix Oscilloscope Camera Model C-27, is presented in Figure 18. The traces are of a 4% asphalt content specimen and a 7% asphalt content specimen. The binder used in the preparation of both specimens was the 85 - 100 penetration grade asphalt. Both traces were made using identical dial settings on the oscilloscope with a horizontal scale setting of 0.1 milliseconds per centimeter. A comparison of the two traces establishes that while the amplitude of the two traces are approximately the same, the initial horizontal portion of the trace for the 7% asphalt content specimen is shorter than the initial horizontal portion of the trace for the 4% asphalt content specimen. This indicates that the "first arrival" of the compressional wave traveling through the 7% asphalt content specimen occurs earlier in time than the "first arrival" of the compressional wave in the 4% asphalt content specimen.

As a final test of the specimens to determine optimum asphalt content for the mixture a standard Hveem Stability (ASTM Number D 1560) Test was performed on two of the specimens of each different asphalt content.

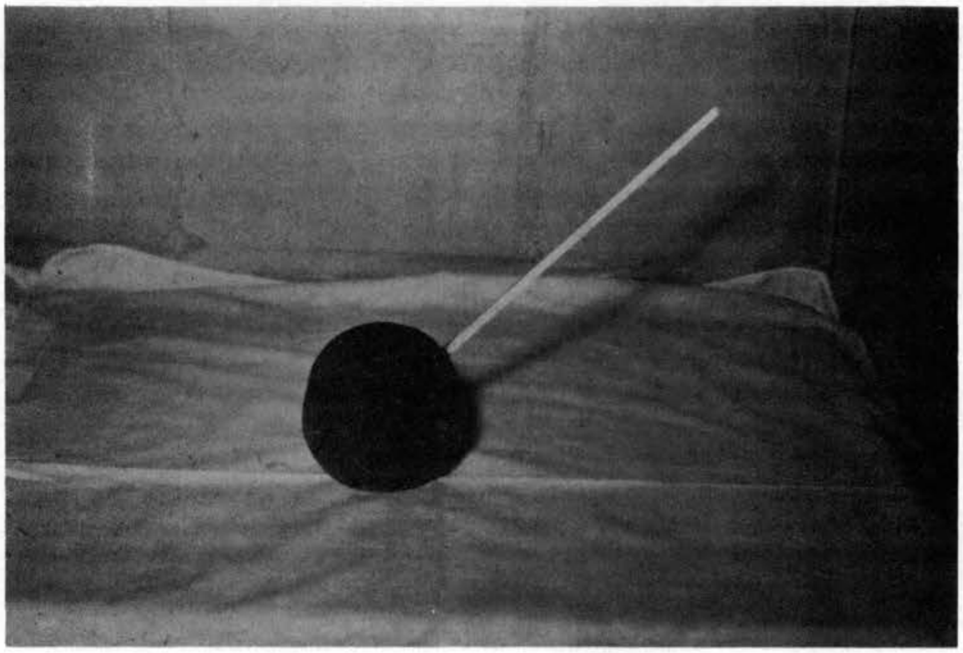
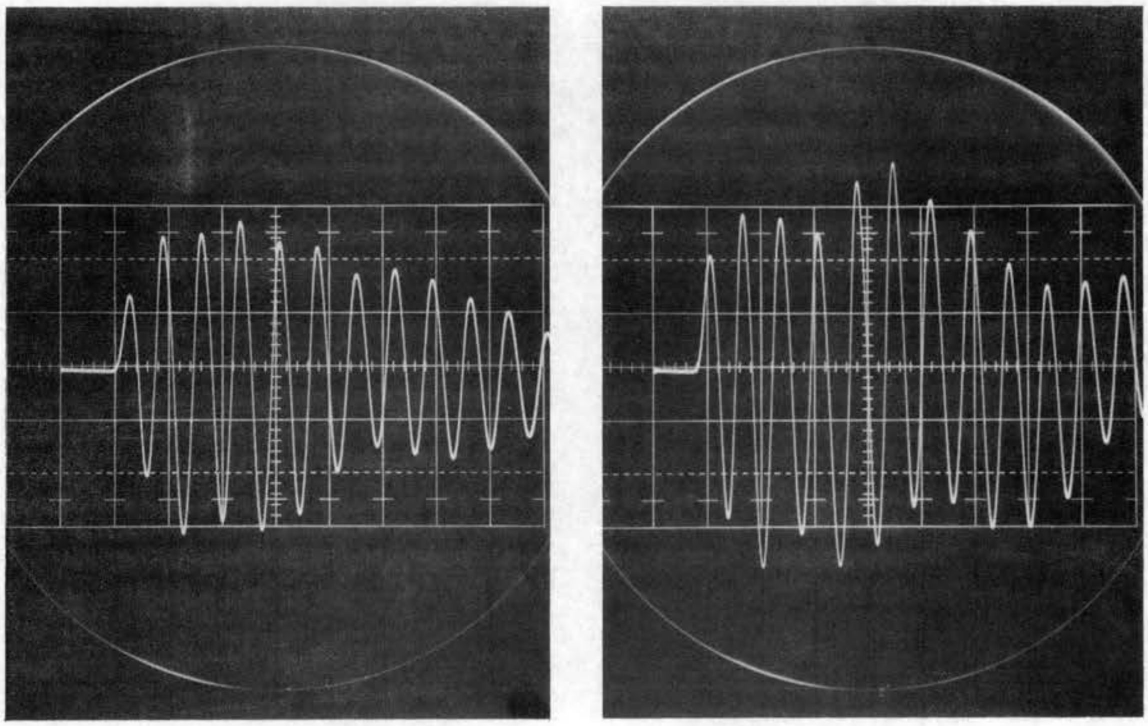


Figure 17. Temperature Monitoring Arrangement



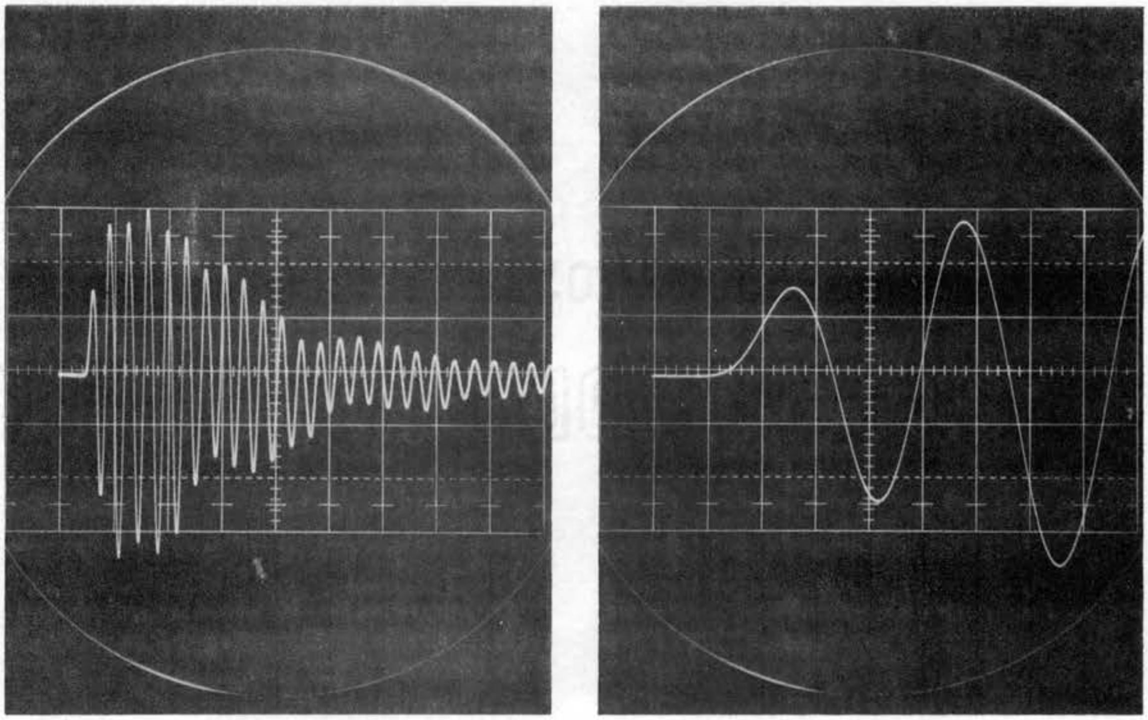
4% Asphalt Content   7% Asphalt Content

Figure 18. Typical Pulse Traces for Specimens of Different Asphalt Contents

Precision

The precision of the compressional wave velocity tests was influenced primarily by the nature of how the "first arrival" appeared on the oscilloscope screen. Instead of a sudden, definite arrival of the wave with a corresponding abrupt and distinct point of vertical deflection of the trace the wave arrived gradually causing the slope of the electron beam to gradually increase as the pulse got stronger. This is shown quite well in Figure 19. Both photographs are of the same specimen with an asphalt content of 4% using the asphalt numbered 2. The upper trace has a horizontal scale of 0.1 milliseconds per centimeter and portrays the first deflection as an abrupt occurrence. However, as seen in the lower trace where the horizontal scale is 10.0 microseconds per centimeter, the electron beam changes slope gradually. It was therefore difficult to determine exactly at what point the "first arrival" occurred. With high amplification the uncertainty of locating the beginning of the "first arrival" could be reduced to about 0.04 microseconds in most cases. Two readings of travel time were made for each specimen with the average of the two used in the wave velocity calculations.

The travel times ranged from 12.35 microseconds to 30.35 microseconds for the asphalt-aggregate specimens. All of the travel time readings could be repeated to ±2 divisions of the multiplier dial. With the settings on the delay time dial ranging from 5 to 10 microseconds per centimeter the maximum variance in time was 0.2 microseconds. This was equivalent to a precision of ±1.5%.



Horizontal Scale = 0.1 milliseconds/cm

Horizontal Scale = 10 microseconds/cm

Figure 19. Nature of "First Arrival" of Compressional Wave Trace

CHAPTER V

RESULTS AND DISCUSSION

Effects of Specimen Size and Shape

According to several investigators (26,27,28) who have used pulse velocity techniques, the initial arrival of the compressional wave (first wave front) travels at the theoretical velocity V_c in materials for which the elastic constants can be determined independently. Using various materials and changing the size and shape of the specimens, they have found that this velocity is independent of length and diameter of the specimen and transducer frequency within certain ranges of dimensions and experimental accuracy.

Birch (27) suggested that in order to keep interference from secondary waves reflected at the material-air interface at a minimum, the diameter to height ratio of the specimen should not exceed about 5. For larger values of the ratio, less energy arrives with the first wave front since more energy is converted to other types of waves at the boundaries of the specimen. The amount of energy converted from one type to another depends on the angle of incidence of the wave and on Poisson's ratio of the material (29).

The specimens used in these tests were molded using the motorized gyratory-shear press (see Chapter IV) and were 4 inches in diameter by approximately 2 inches in height. It was considered that this molding procedure yielded specimens of more uniform density than impact or static methods of compaction. Since the diameter to height ratios of these specimens were approximately 2, they were well within Birch's suggested limit of 5. In addition, this size of specimen is standard for the Hveem-Gyratory testing procedure and the specimens could be subjected to further tests involved in this method of mix design. This is a distinct advantage of the pulse velocity technique of dynamic testing over the resonant frequency method.

Data Reduction

A great many determinations of travel time were made under the various test conditions. In order to reduce these readings to compressional wave velocities, a computerized solution procedure was developed. The use of an IBM 7040 computer reduced substantially the amount of time required to compute the velocities and also gave highly precise results. The Fortran IV Language program was developed so that it could be applicable to future work as well as the present study. An important feature of the program is that the output material is arranged for easy interpretation. A flow chart and a program listing is presented in Appendix A.

Temperature Effects

In order to study the effects of temperature on the behavior of the compressional wave velocity in the asphalt-aggregate mixtures, a plot of the velocity in feet per second as the ordinate, against temperature in degrees fahrenheit as the abscissa, was made for each penetration series of specimens at the various asphalt contents. Although there was a considerable amount of scatter in the data, there did seem to exist a linear relationship between compressional wave velocity and temperature for three distinct segments of the curve. These segments were from -9° F to approximately 35° F, from approximately 35° F to approximately 130° F and from approximately 135° F to 170° F. Because of the suggested linear relationship between velocity and temperature of the mixture, a linear regression analysis was applied to the three segments of the respective curves. The temperature ranges corresponding to these linear segments were -10° to 30° F, 40° to 120° F and 140° to 170° F.

Typical plots of the data for 5% and 7% asphalt content specimens are presented in Figure 20. These curves show that while there is a scattering of data points, the linear regression curves fit the data fairly well. The data scatter can be attributed to several sources. Non-homogeneity of the aggregate particle sizes and orientation in the compacted specimens is, perhaps, the most obvious reason for this. Other factors influencing the scatter are: 1) lack of ability to determine the exact temperature of each specimen at the time of test, 2) difficulties in determining the exact point on the trace at which "first arrival" of the compressional wave occurs, and 3) random errors inherent in the testing procedure.

Over the total temperature range shown in Figure 20, the compressional wave velocity in the specimens decreased as temperature increased although the rate of decrease was quite different within the three aforementioned ranges. Figure 20 also shows that for a given grade of asphalt cement the slopes of the lines for specimens at various asphalt contents are essentially parallel within each of the three temperature ranges. This tendency held for all specimens made with both grades of

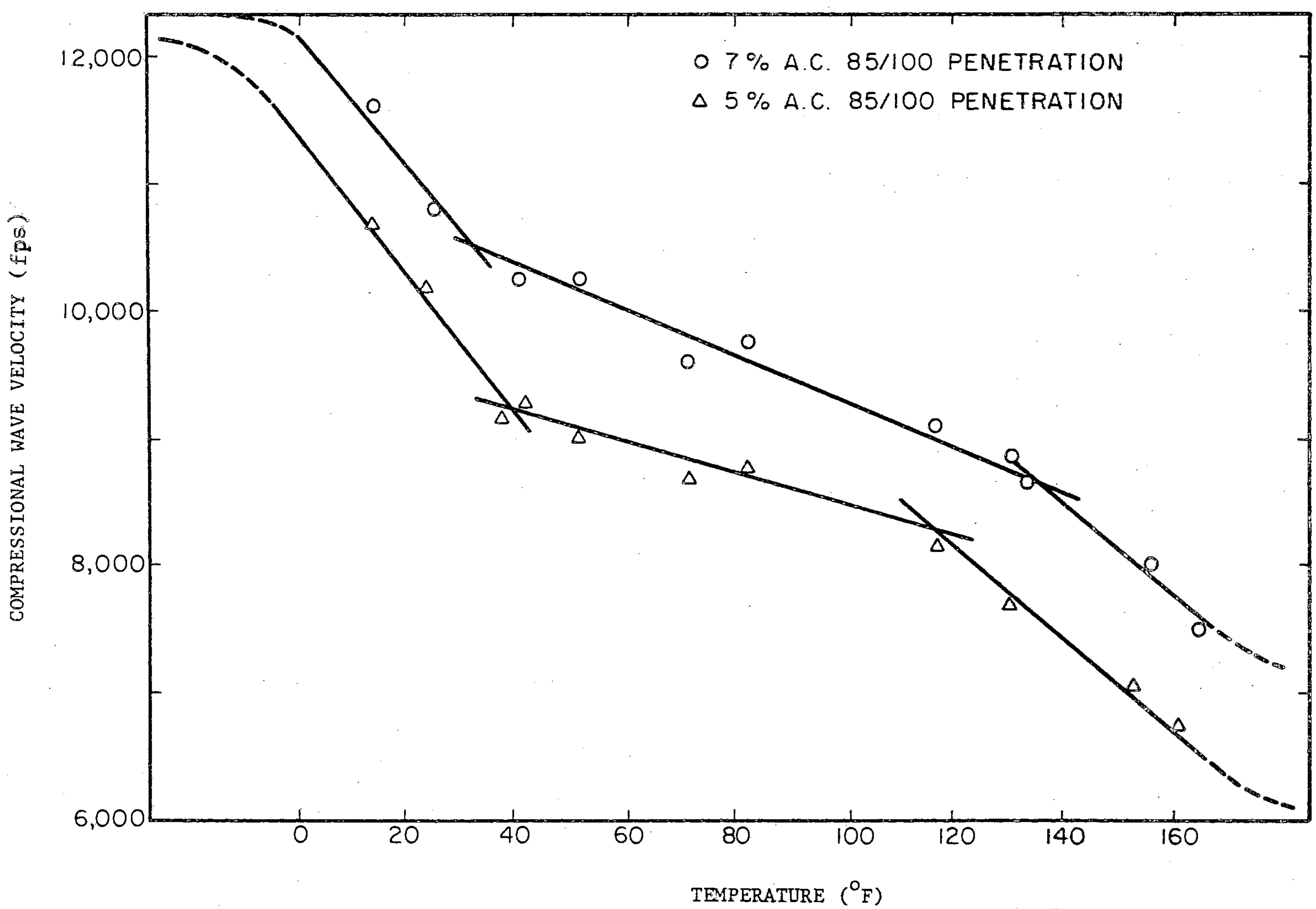


Figure 20. Typical Plots of Compressional Wave Velocity Versus Temperature

asphalt cement and suggests that this is a behavior pattern characteristic of a given asphalt-aggregate mixture.

The points of intersection of the three segments of the curves occurring at approximately 40° F and 125° F were determined by extrapolation of the regression lines. Little significance can be attached to these slope change points, however, the points of intersection for all asphalt contents were within the temperature range of 30° to 40° F at the lower point of change and between 120° to 140° F at the upper point of change.

The slope change that occurs within these two narrow ranges of temperature is quite significant. For all the specimens tested, the average change in slope occurring in the 30° to 40° F range is approximately 30% while the average change in slope occurring in the 120° to 140° F range is approximately 21%. This indicates that these relatively narrow limits are transitional temperature ranges within which the asphalt-aggregate mix undergoes rather rapid changes in dynamic properties. By careful measurement within these transitional ranges it should be possible to ascertain whether these property changes take place gradually or are a more or less abrupt phenomenon.

For comparative purposes, linear regression line plots were made for the 60-70 and 85-100 penetration asphalt cement specimens within each of the previously mentioned temperature ranges. The more or less linear relationships between temperature and the velocity exhibited by the mixtures is indicative of separate and distinct types of dynamic behavior within these ranges.

The behavior of an asphalt-aggregate mixture is complicated by its deformation characteristics at various temperatures related to the viscosity of the asphalt binder. At relatively high temperatures the mixture may be a highly plastic (tending to viscous) material and at lower temperatures the mixture may be considered as an elastic material. Between these temperature extremes the material will probably exhibit both elastic and plastic characteristics.

In the low temperature range (see Figures 21 and 22) the linear regression lines are remarkably parallel indicating similar behavior of the asphalt-aggregate mixtures regardless of the percentage of asphalt incorporated in the specimens. The relatively steep slopes of the lines in this range show that the wave velocity is greatly influenced by temperature. In Figure 21, the decrease in velocity per 1° F rise in temperature ranged from 41.0 fps to 56.7 fps. The 85-100 penetration asphalt cement specimens (Figure 22) behaved similarly with velocity decreases per 1° F rise in temperature ranging from 35.2 fps to 53.7 fps. Generally, the rate of change of velocity with temperature increased with increase in asphalt content for the 60-70 penetration series of specimens but this did not hold true for the 85-100 penetration series. It is suggested that the behavior of the specimens within this range of temperature is essentially elastic in nature.

As temperature is decreased the viscosity of an asphalt cement will increase, the material characteristics changing from a viscous liquid to a semi-solid and finally to a brittle solid. In an asphalt-aggregate mixture the films of asphalt surrounding the aggregate particles serve as a binder or cementing agent. As the nature of the films change with decreasing temperature, the aggregate matrix is more tightly bound together, i.e., it becomes more rigid. Consequently, the mixture is able to transmit the compressional wave at a much higher velocity.

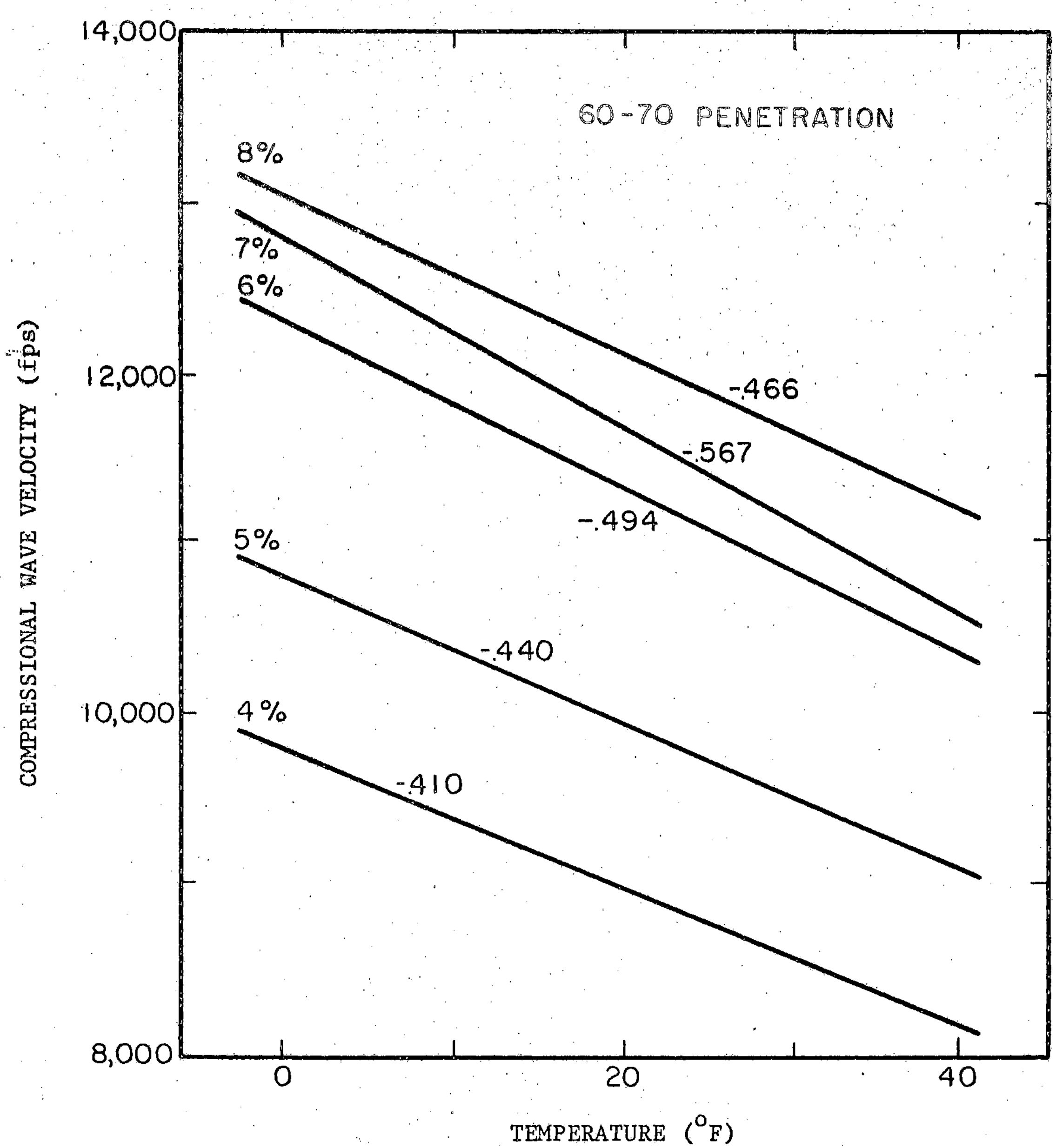


Figure 21. Low Temperature Range Regression Lines - 60 to 70 Penetration Specimens

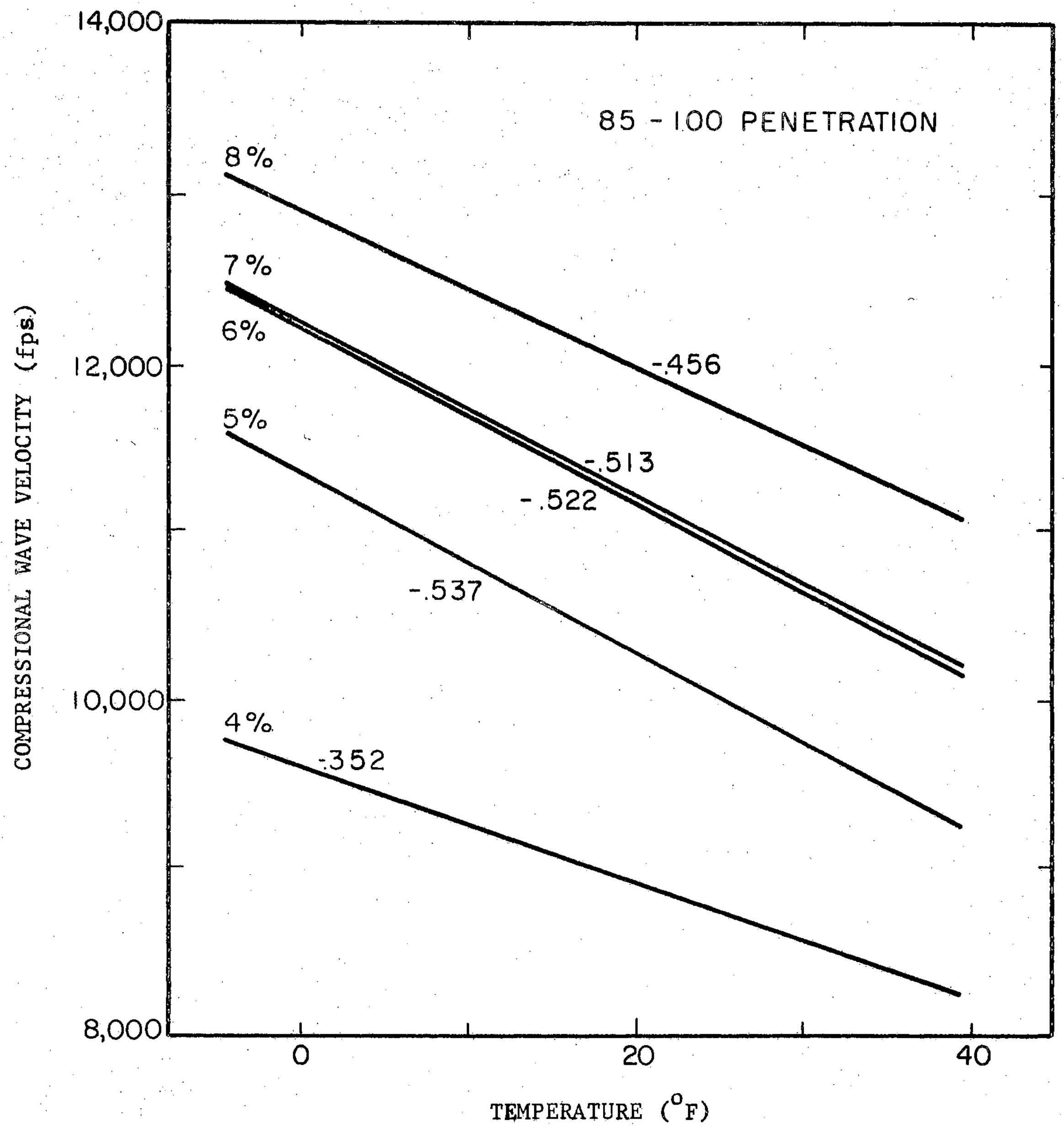


Figure 22. Low Temperature Range Regression Lines - 85 to 100 Penetration Specimens

The wave velocity versus temperature plots for the high (130^{o} to 170^{o} F) temperature range (Figures 23 and 24) are very similar to the plots for the low temperature range in that the slopes of the lines within these ranges are quite comparable in magnitude. The decrease in velocity per 1^{o} F rise in temperature for the 60-70 penetration asphalt cement varied from 32.8 to 50.9 fps while for the 85-100 penetration asphalt the decrease ranged from 28.7 to 52.3 fps. Again the slopes of these curves are fairly parallel indicating similar behavior of the various specimens tested. Notice that crossing of the linear regression lines for the higher asphalt contents occurs in both plots. As in the low temperature range, the tendency toward an increasing slope of the lines with increasing asphalt content can be observed. This was true in all cases except the 7%, 85-100 penetration lines. The behavior of the specimens in this temperature range is believed to be indicative of the plastic nature or characteristics of the mixture.

Within the middle ranges of temperatures (Figures 25 and 26) the lines show a marked decrease in slope in comparison with those in Figures 21 and 22 as well as the curves in Figures 23 and 24. The flatter slopes indicate a reduction in the effect of temperature on the compressional wave velocity. In Figure 25 the average reduction in velocity per degree Farenheit for the 4-7% specimens is about 13.5 fps. For the 85-100 penetration specimens (Figure 26) the average reduction in velocity for the 4-7% asphalt contents is about 15.4 fps per degree Farenheit. In both figures the 4-7% lines are roughly parallel while the 8% lines have much greater slopes than those for the lower asphalt contents. If, as hypothesized earlier, this behavior is a combination

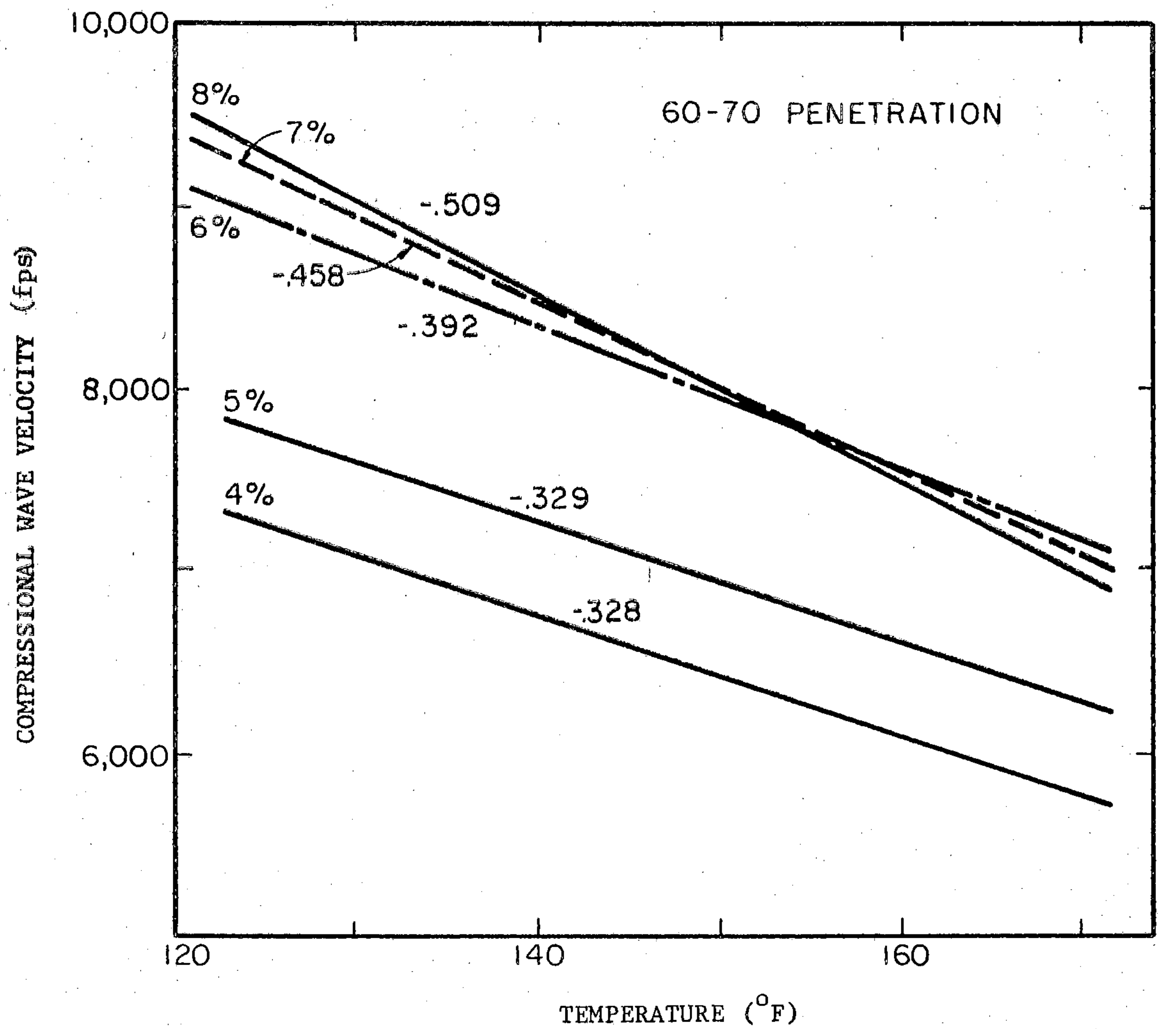


Figure 23. High Temperature Range Regression Lines - 60 to 70 Penetration Specimens

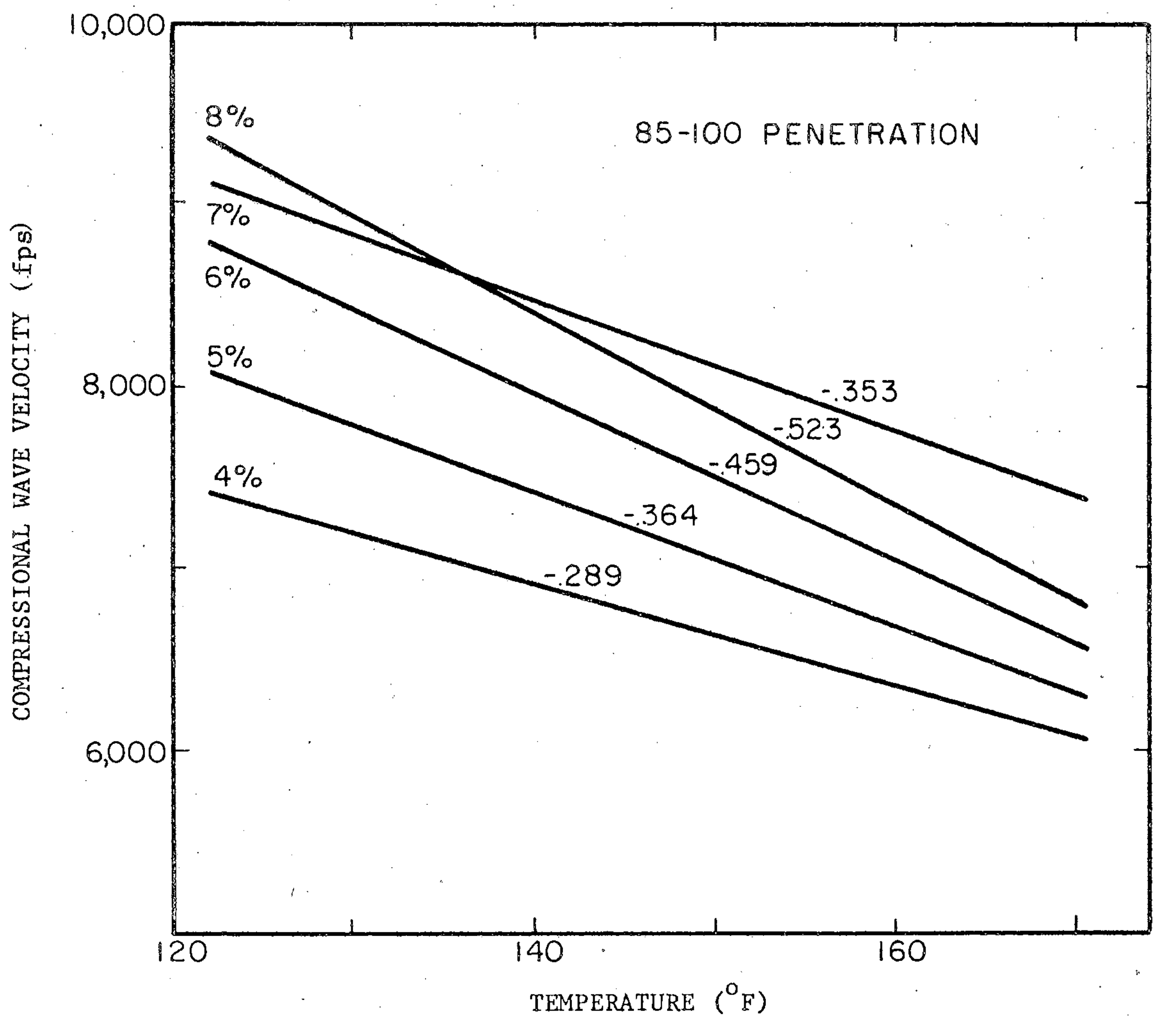


Figure 24. High Temperature Range Regression Lines - 85 to 100 Penetration Specimens

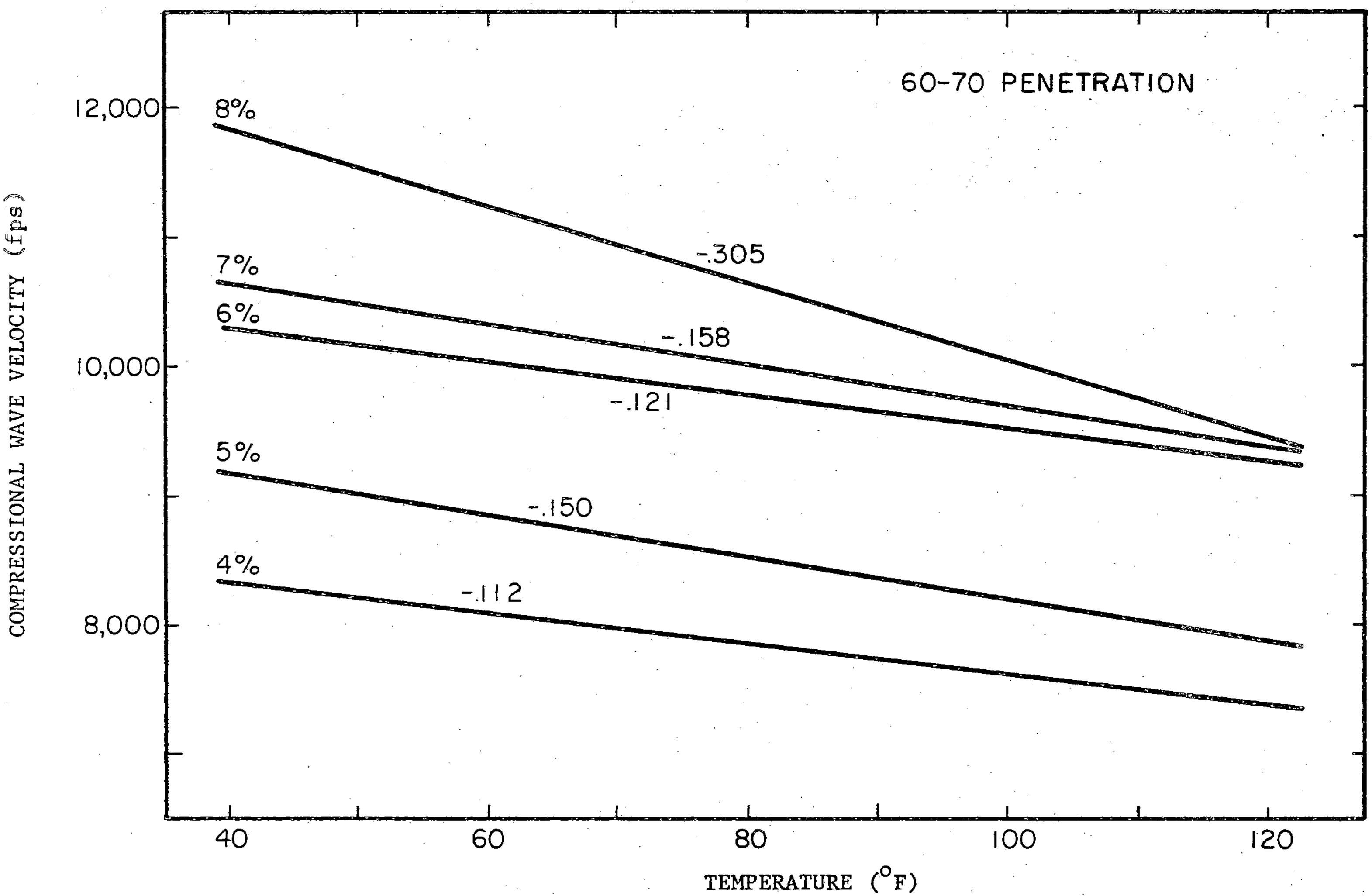


Figure 25. Middle Temperature Range Regression Lines - 60 to 70 Penetration Specimens

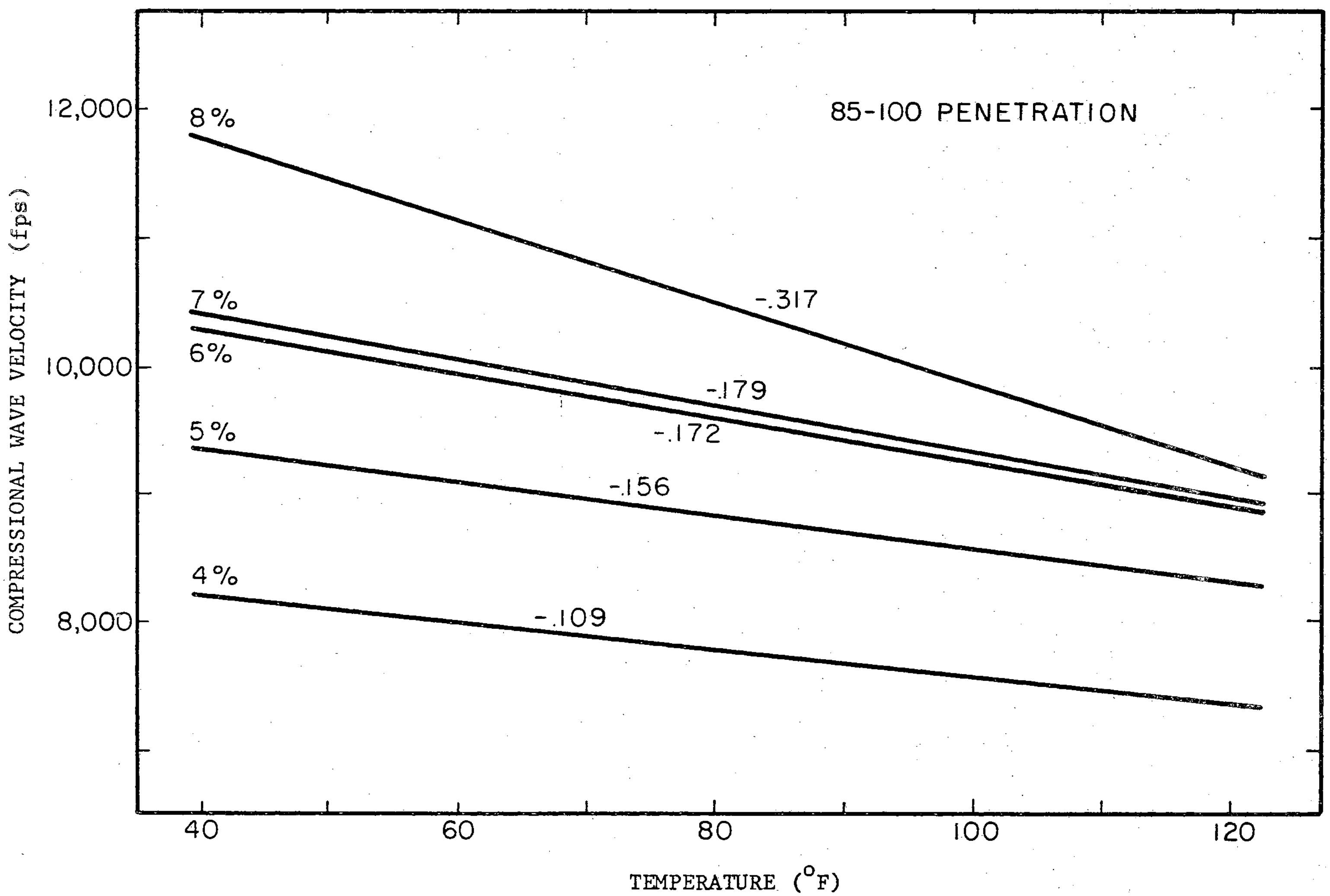


Figure 26. Middle Temperature Range Regression Lines - 85 to 100 Penetration Specimens

of that occurring on either side of this temperature range the behavior within this middle range could be described as elasto-plastic in nature.

Effects of Asphalt Content

With few exceptions, the slopes of the lines within a given temperature range increased with increasing asphalt content. This behavior can be related directly to the asphalt constituent in the mixture.

As stated previously, the incorporation of asphalt into the mixture results in a coating of the aggregate particles with the material. This coating serves as a binder holding the aggregate particles together in the matrix. At low asphalt contents, the film of asphalt covering each aggregate particle is thin thereby allowing adjacent particles to be in intimate contact with each other. As the asphalt content of the mix is increased these relatively thin asphalt films increase in thickness hence reducing the amount of intra-particle contact. This reduction in contact between the particles of aggregate in the mixture results in the mixture's behavioral characteristics becoming more and more dependent upon the characteristics of the asphalt cement.

In the previously presented figures it is obvious that at a given temperature the compressional wave velocity also increases with increased percentage of asphalt content in the mixture. It was expected that this relation would hold until the optimum asphalt content for the mixture was reached and that increases in asphalt content above the optimum quantity would result in a decrease in the compressional wave velocity.

When an asphalt-aggregate mixture is compacted there will be incorporated in the specimen a certain quantity of air voids. As the amount of asphalt is increased in the specimen, the amount of these air voids will decrease to some minimum value. Since the compressional wave is mechanical in nature, it cannot travel through the air voids and must seek out a path around the voids via the aggregate particles and the asphalt binder. As the amount of air voids is reduced, fewer by-passes will have to be made by the wave and the smaller the travel time will be through the specimen. However, when the optimum asphalt content is surpassed, the excess asphalt tends to force the aggregate particles apart, reducing the inter-granular contact. This causes the compressional wave to be transmitted more through the asphalt constituent than the more rigid aggregate matrix, thereby increasing the travel time of the wave and greatly reducing its velocity.

Other researchers in the field such as Manke and Galloway (1) have indicated that the compressional wave velocity does, in fact, decrease in specimens whose asphalt contents are above optimum. Hutcheson (29) also found that the maximum compressional wave velocity of various asphalt-aggregate combinations corresponded closely with the optimum asphalt contents as determined by the Marshall Method.

The optimum asphalt contents for the specimens in this study were determined from Hveem-Gyratory test data. The optimum content for the 60-70 penetration grade specimens was 6½%. A definite optimum value could not be fixed for the 85-100 penetration grade specimens, but since the aggregate gradation was the same for both series of specimens it is reasonable that the optimum asphalt content for this mix would be somewhat less than for the mix containing the harder consistency asphalt cement.

The test results of this study seem to contradict the above discussed relationship between wave velocity and asphalt content, in that the wave velocity continued to increase with increased asphalt content above the optimum values for the mix as suggested by the Hveem-Gyratory tests. There should exist, however, an "optimum" asphalt content where a maximum compressional wave velocity for the mixture will occur. This "optimum" value is not necessarily related to the optimum asphalt content as determined by standard procedures. At temperatures above about 150° F the 60-70 penetration specimen velocities (Figure 23) for the 7 and 8 % asphalt contents are reduced below that of the 6% content specimens. A similar trend is shown in Figure 24 where the 8% content specimens exhibit lower velocities than the 7% specimens at temperatures above about 135° F.

No direct correlation of optimum asphalt content and maximum compressional wave velocity could be made. More study of this relationship is needed along with a more detailed investigation of the effects of air void content on the compressional wave velocity.

Effects of Type of Asphalt Cement

Figure 27 is a typical plot in the low temperature range showing the effect of penetration grade asphalt cement on compressional wave velocity for 4 and 7 % asphalt content specimens. As indicated, the specimens made from harder asphalt cement exhibited higher velocities. This relationship did not hold true in all cases for each of the three temperature ranges, however, it is believed that the data scatter encountered would account for most of these deviations and that generally this relationship would be valid. However, the temperature

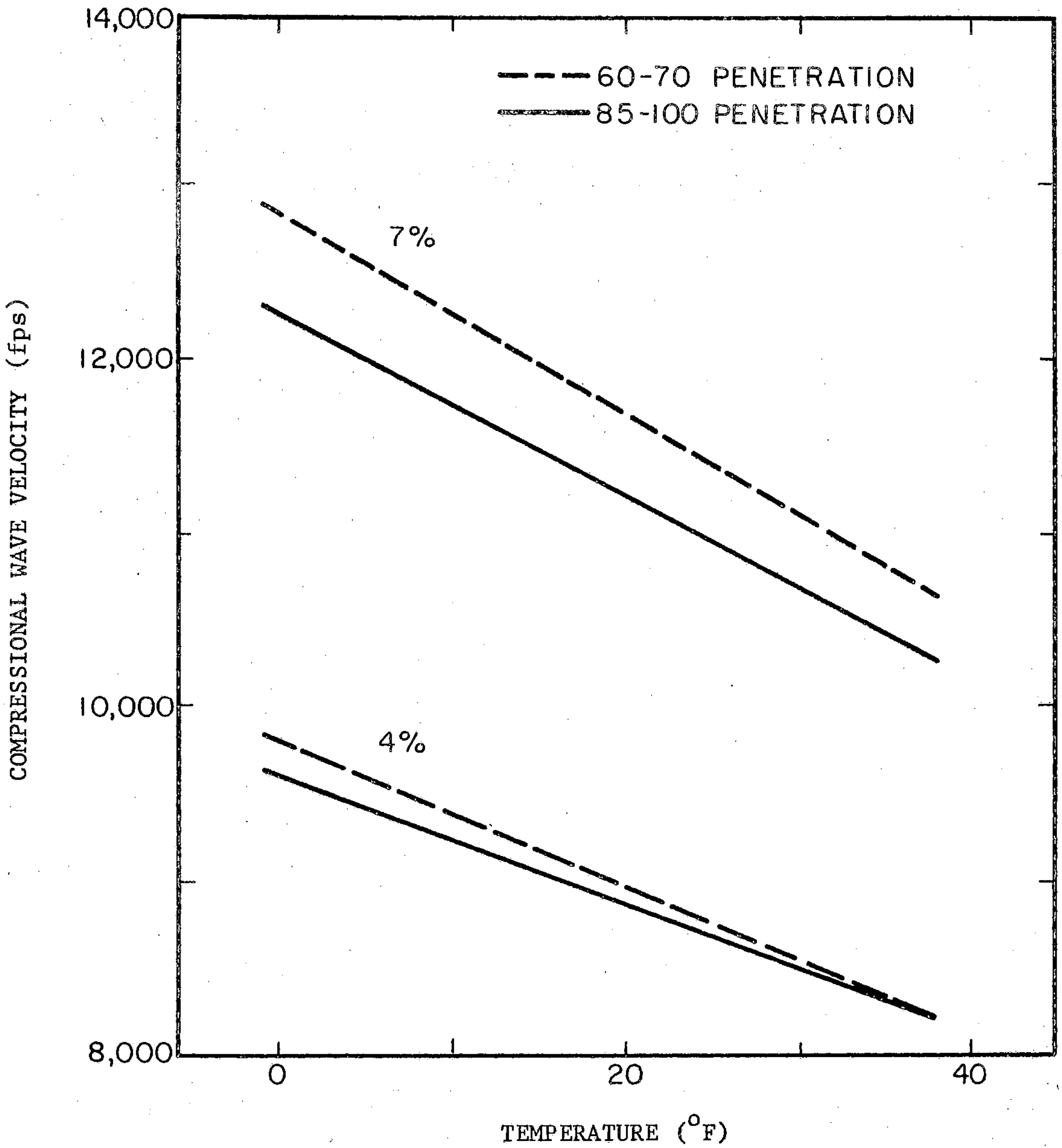


Figure 27. Effects of Penetration Grade on Compressional Wave Velocity

susceptibility characteristics of the asphalt cement will greatly influence the results, i.e., the incremental decrease in velocity with increased temperature will depend on the manner in which the viscosity of the asphalt cement decreases.

Pulse Moduli

To examine the effect of Poisson's ratio on the E-modulus of asphalt mixtures, moduli values were calculated from equation 3.6 for the temperature range from 36° F to 121° F. These values have been termed "pulse moduli" (E_p), relating to their determination from pulse velocity measurements, and are given in Table V. These E_p values were calculated using an average density, an average V_c and assuming two different Poisson't ratios (μ). As seen in Table V a decrease in Poisson't ratio from 0.4 to 0.3 results in an increase of approximately 60% in the pulse moduli values at a given temperature. This indicates that the value of μ is critical and should not be assumed for an accurate evaluation of the elastic properties of an asphalt-aggregate mix. The calculated values of E_p were of the same degree of magnitude as the sonic moduli of bituminous mixtures determined by Goetz (2).

As indicated in Table V the pulse modulus varies significantly within the middle range of temperatures. For the 5% specimens, there was approximately a 28% reduction in E_p from 41° F to 121.5° F and for the 7% specimens the reduction in E_p was about 24%. The percent reduction was about the same for both assumed values of Poisson's ratio. Since there is a direct relationship between compressional wave velocity and E-modulus this reduction in E-modulus with increasing temperature would be greater in the 0 to 40° F and the 130° to 170° F temperature ranges.

Present flexible pavement design procedures utilize E-modulus values to estimate the required thickness of bituminous paving mixtures. A more realistic E_p or E-modulus value for design purposes could be obtained with the pulse velocity technique by determining the middle behavioral temperature range of a given asphalt-aggregate mixture and using either an average E_p value or an E_p for a median temperature within this range.

TABLE V

PULSE MODULI VALUES (E_p) FOR 85-100

PENETRATION ASPHALT SPECIMENS

Temperature (^oF)	E_p (psi x 10^6)			
	5% A.C.		7% A.C.	
	$\mu = 0.3$	$\mu = 0.4$	$\mu = 0.3$	$\mu = 0.4$
41.0	1.821	1.140	2.176	1.366
50.0	1.752	1.099	2.195	1.378
62.5	1.616	1.014	2.141	1.344
69.5	1.751	1.098	———	———
75.0	———	———	2.153	1.352
81.0	1.644	1.031	2.029	1.274
105.0	1.391	0.872	1.701	1.068
121.5	1.313	0.824	1.647	1.034
% Reduction in E_p	27.9%	27.7%	24.3%	24.3%

CHAPTER VI

CONCLUSIONS

This research was a portion of a study of the dynamic elastic properties of asphalt-aggregate mixtures. The primary objectives of this work were to evaluate the compressional wave response of asphalt-aggregate mixtures at various temperatures and, using this information, check the possibility of defining behavioral temperature ranges as well as the zone of transition where the dynamic properties of the mixtures undergo a change. It is believed that these objectives were accomplished and that because of this, the way has been cleared for further work in this area.

Taking into consideration the materials and procedures used, the following conclusions may be drawn:

1. Between -10° F and $+170^{\circ}$ F there are three distinct ranges of temperature within which the asphalt-aggregate mixtures exhibit different linear relationships between compressional wave velocity and temperature, i.e., the slope of the curve changes drastically from one range to another.

2. The slope change points for all mixtures occurred between 30° and 40° F and between 120° and 140° F. These relatively narrow ranges of temperature are considered transitional temperature ranges within which the dynamic properties of the asphalt-aggregate mixture undergo a change.

3. Within these three temperature ranges, the slope of the curve will increase with increased asphalt content.

4. Generally, an increase in asphalt content increases the compressional wave velocity at a given temperature. This is expected to hold until <u>some</u> optimum asphalt content for the mixture is achieved. However, this optimum content does not necessarily correspond to the optimum asphalt content as determined by standard methods.

5. Poisson's ratio has an appreciable influence on pulse modulus values and therefore should not be assumed for accurate evaluation of the elastic properties of the mixture.

6. For a given asphalt content, mixtures containing a harder consistency asphalt will have a higher compressional wave velocity. However, this relationship is dependent to a considerable extent upon the temperature susceptibility of the asphalt cement.

7. The pulse modulus of asphalt-aggregate mixtures varies considerably within the ranges of temperature normally expected in the field.

CHAPTER VII

SUGGESTIONS FOR FURTHER INVESTIGATION

This investigation has delineated several areas in which further studies are needed. It is suggested that further studies in this field should include the following:

1. The narrow ranges of temperature within which the properties of the mix undergo rapid change should be examined closely to clarify the behavior of the material properties in these ranges.

2. A study should be undertaken to determine if maximum wave velocity in the specimen is a function of optimum asphalt content or primarily a function of the void ratio of the compacted mixture.

3. An extension of the tests to temperatures both above and below those examined in these tests would be worthwhile.

4. Apparatus and techniques should be developed by which both shear and compressional wave velocities in the material could be obtained. This would enable direct determination of Poisson's ratio and E_p values for the mixtures.

5. The effects that aggregate size, shape, and gradation used in the mixture, have upon the wave velocity should be examined.

6. An analysis of the effects of both age-hardening and cyclic freezing and thawing of asphalt-aggregate mixtures upon the wave velocity should be undertaken.

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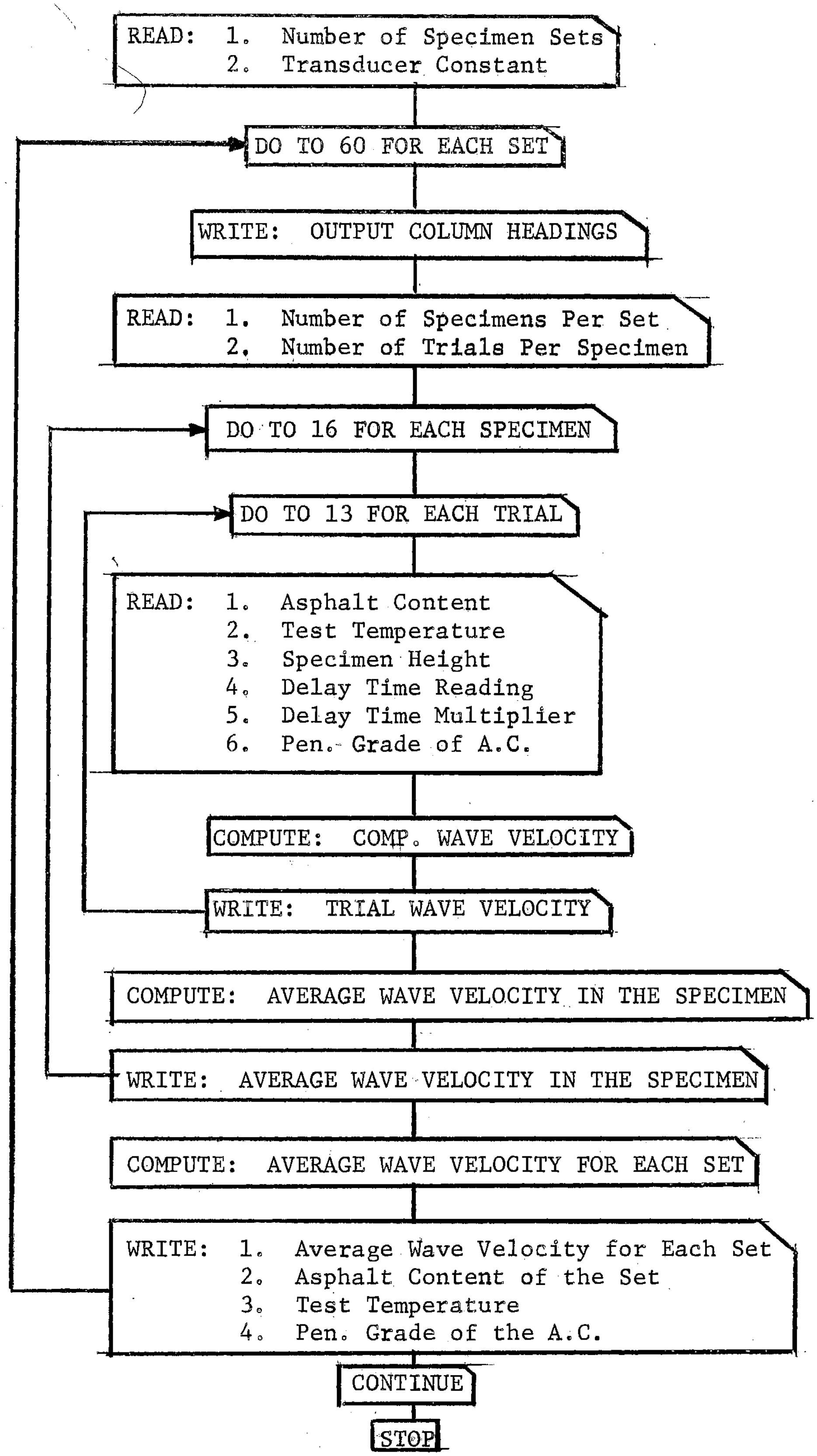
APPENDIX

COMPUTER PROGRAM FOR DATA REDUCTION

SUMMARY FLOW DIAGRAM



PROGRAM LISTING

```
C     PROPERTIES OF ASPHALT MIXTURES
C     85 TO 100 PENETRATION
      DIMENSION V(300), TIME(300),BMUL(300),VIPS(300),VFPS(300),AN(300)
     4,BM(300)
    2 READ (5,200) LSET,CONST
    3 DO 60 K = 1,LSET,1
      WRITE (6,900)
    4 VSUM = 0.
    5 READ (5,100)  NUM,MTRIL
    6 DO 50 I = 1, NUM,1
    7 VTOT = 0.
    8 DO 40 J = 1,MTRIL,1
    9 READ (5,400) KAC,TEMP,HT,TIME(J),BMUL(J),AN(K),BN(K)
   10 TIME(J) = (TIME(J) *BMUL(J) - CONST) * 10.**(-6.)
   11 HT = HT/2.54
   12 VIPS(J) = HT/TIME(J)
   13 VFPS(J) = (VIPS(J))/(12.)
      WRITE (6,901)I,J,VIPS(J),VFPS(J)
   40 VTOT = VTOT + VFPS(J)
      TRIAL = MTRIL
   15 V(I) = VTOT/TRIAL
   16 VSUM = VSUM + V(I)
   50 WRITE (6,600)V(I)
      ANUM = NUM
   17 VAVG = VSUM/ANUM
   18 WRITE (6,700) VAVG
   60 WRITE (6,300) KAC,TEMP,AN(K),BN(K)
  100 FORMAT(I5,5X,I8)
  200 FORMAT(I5,5X,F10.2)
  300 FORMAT(16X,I5,1X,23HPERCENT ASPHALT CONTENT,5X,17HTEMPERATURE(F)=
     1 ,F7.2,5X,2A5 ,1X,11HPENETRATION/////)
  400 FORMAT (I5,F10.2,F10.2,F10.2,F10.2,2A5)
  600 FORMAT (  26X,28HAVERAGE SPECIMEN VELOCITY = ,F10.2,/ )
  500 FORMAT(2X,16HVELOCITY(IPS) = ,1PE10.3,5X,16HVELOCITY(FPS) =
      ,1PE10 2.3)
  900 FORMAT (20X,8HSPECIMEN,5X,5HTRIAL,5X,13HVELOCITY(IPS),
     35X,13HVELOCITY(FPS)
  901 FORMAT (23X,I2,9X,I2,9X,1PE10.3,7X,1PE10.3)
  700 FORMAT (26X,29HAVERAGE WAVE VELOCITY(FPS) = ,F10.2,/)
   19 CONTINUE
   20 STOP
      END
$ENTRY
```

VITA

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